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Coastal and Hydraulics Laboratory

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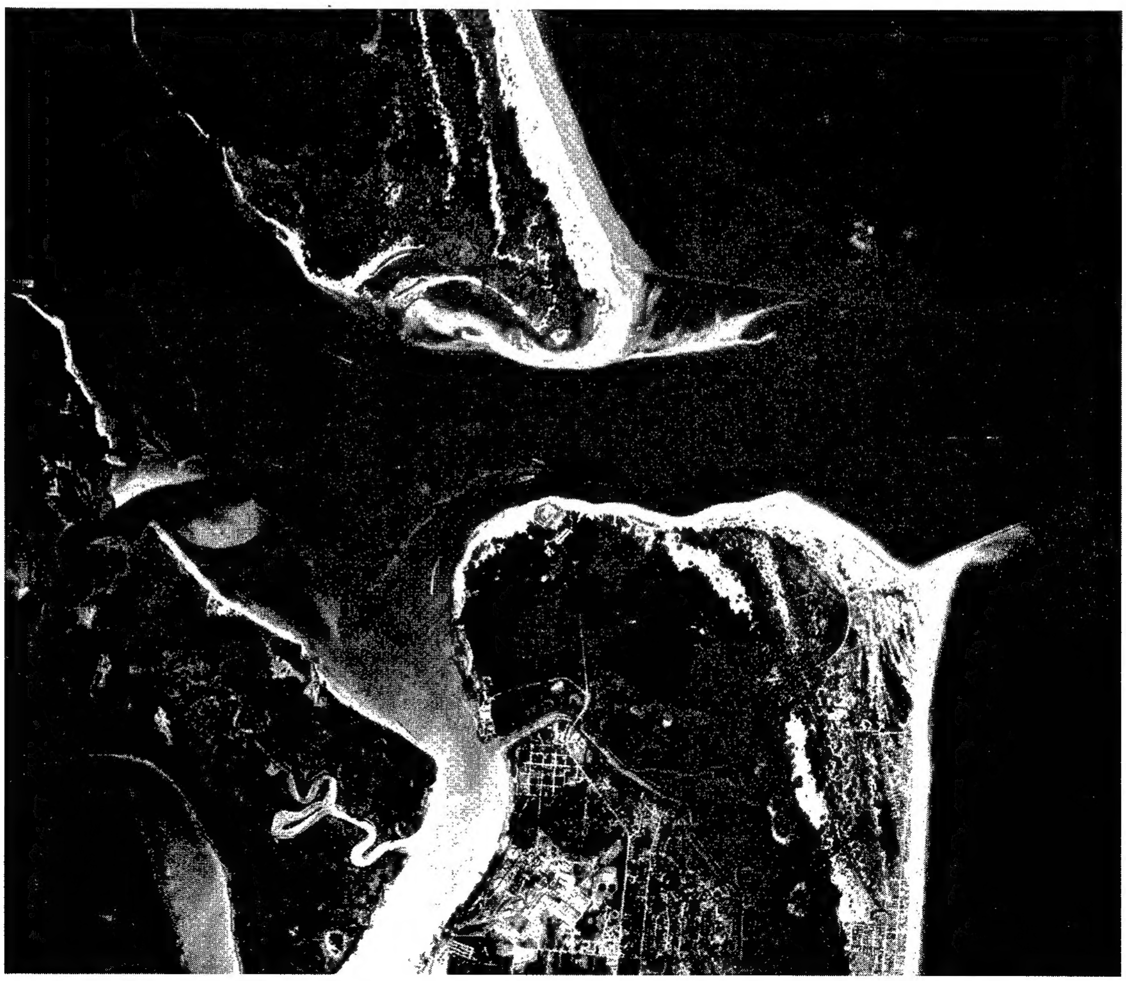
DMS: Diagnostic Modeling System

Report 4
Shoaling Analysis of St. Marys Entrance, Florida

Shelley Johnston, Nicholas C. Kraus, Mitchell E. Brown, and William G. Grosskopf

September 2002



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Shoaling Analysis of St. Marys Entrance, Florida

by Shelley Johnston, Nicholas C. Kraus, Mitchell E. Brown

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Report 4 of a series

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Under Work Unit 33129

Contents

List of Figures

List of Tables

Preface

This report presents an analysis of the sediment shoaling characteristics of St. Marys Entrance, Florida, performed for the U.S. Army Engineer District, Jacksonville. Diagnostic Modeling System (DMS) procedures were applied to determine causes of channel shoaling and evaluate possible mitigation alternatives to reduce the frequency and cost of dredging. This study was performed under the DMS Work Unit No. 33129, of the Coastal Sedimentation and Dredging Program administered by Headquarters, U.S. Army Corps of Engineers (HQUSACE). Research and Development activities of the DMS are being conducted at the U.S. Army Engineer Research and Development Center (ERDC), Coastal and Hydraulics Laboratory (CHL), Vicksburg, MS. HQUSACE Program Monitors are Messrs. Charles B. Chesnutt and Barry W. Holliday.

Work was performed and this report written by Ms. Shelley Johnston, Coastal Evaluation and Design Branch (CEDB), CHL; Dr. Nicholas C. Kraus and Mr. Mitchell E. Brown, Senior Scientist Group, CHL; and Mr. William G. Grosskopf, Offshore & Coastal Technologies, Inc.-East Coast, Avondale, PA. The Principal Investigator of the DMS research unit was Dr. Kraus. Assistance from the Jacksonville District was provided by Mr. Brian K. Brodehl, Ms. Christina Brusnahan, Mr. Daniel W. Beasley, Mr. Tom R. Martin, and Mr. Francis M. Woodward. The Jacksonville District project monitor was Mr. Beasley. Ms. J. Holley Messing, CEDB, was responsible for the final formatting of this report.

Work within CEDB was performed under the supervision of Ms. Joan Pope and Dr. Yen-Hsi Chu, former Chief and Chief, respectively, CEDB. This project was performed under the administrative supervision of Mr. Thomas W. Richardson, Director, CHL.

At the time of publication of this report, Dr. James R. Houston was Director of ERDC, and COL John W. Morris III, EN, was Commander and Executive Director.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
cubic feet	0.0283168	cubic meters
cubic yards	0.7645549	cubic meters
feet	0.3048006	meters
knots (international)	0.5144444	meters per second
miles (U.S. statute)	1.6093472	kilometers
square feet	0.09290304	square meters

1 Background and Problem Statement

Introduction

This study on sediment shoaling in St. Marys Entrance, Florida, was performed by the U.S. Army Engineer Research and Development Center (ERDC), Coastal and Hydraulics Laboratory (CHL), for the U.S. Army Engineer District, Jacksonville. The purpose was to apply Diagnostic Modeling System (DMS) methodology (Kraus 2000; Kraus and Taylor 2000; Kraus et al. in preparation) to St. Marys Entrance in response to the Jacksonville District's interest in reducing operation and maintenance costs for the entrance navigation channel.

The DMS approach relies on systematic analysis of qualitative and quantitative data, together with reference to analogues of similar physical systems. Numerical simulations of coastal and tidal processes are typically conducted as an adjunct or to provide further information for confirming or complementing the empirical approach. Much of the analysis and data archival tasks are housed in the DMS Data Manager (Craig et al. 2001). The aim is to arrive at feasible alternatives for reducing dredging within a typical maintenance dredging cycle.

Approximately 592,000 cu yd[1] of maintenance dredging is performed annually at St. Marys Entrance Channel. Of this volume, 70 percent (mainly silt and clay) is disposed offshore and 30 percent (mainly sand) is placed on the downdrift beach or in the nearshore. Persistent sediment shoaling of the entrance channel occurs in certain sections. Wideners are maintained along two stretches of the channel by the Jacksonville District for the U.S. Navy. The wideners are designed to trap sediment outside the channel template, thereby reducing the frequency of maintenance dredging while maintaining navigable depth in the authorized channel. Some opportunity also exists to place material on the beach

[1] This study involves analysis of historic and recent engineering documents and data sets with values expressed in U.S. Customary (non-SI) units. To maintain continuity with the previous body of work and ongoing engineering practice, the original units are retained in their context. A table of factors for converting non-SI units of measurement to SI units is presented on page x. Measurements of the oceanographic quantities of waves, water level, and current are expressed in SI units.

fronting Fort Clinch, Florida (Olsen 2001). Recent shore protection work has rehabilitated and modified an existing groin field to protect a historically significant structure at Fort Clinch. The Jacksonville District has received a request to supply dredged sand to backfill the groin field.

St. Marys Entrance provides access from the Atlantic Ocean to the Port of Fernandina, Kings Bay Naval Base, and the Atlantic Intracoastal Waterway (AIWW) for commercial and military vessels. The Port of Fernandina has been active since the 1800's. Today the port handles cargoes from container ships, general cargo ships, refrigerated vessels, and specialized container ships (Raichle, Bodge, and Olsen 1997). The United States military has been present in this area since 1816. Today, reliable transit of *Ohio*-class submarines, commonly referred to as Trident submarines, dictates the controlling depth and much of the channel maintenance.

Description of Site

St. Marys Entrance connects Cumberland Sound to the Atlantic Ocean. The inlet is on the boundary of Florida and Georgia (Figure 1). Three comprehensive publications document the history of St. Marys inlet: Olsen (1977), Kraus, Gorman, and Pope (1994), and Raichle, Bodge, and Olsen (1997). It is not known when the inlet opened, but it appears on maps as early as 1770.

The Fernandina Beach tide station located in Cumberland Sound is the closest National Oceanic and Atmospheric Administration (NOAA), National Ocean Service (NOS) long-term tide station to St. Marys Entrance (Figure 2). Characteristics of tidal water level in the vicinity of the site are reviewed by Kraus, Faucette, and Rogan (1997). The difference between mean higher high water (mhhw) and mean lower low water (mllw) at the Fernandina Beach tide station is 2.01 m. The NOS placed two temporary tide gauges within St. Marys Entrance (Figure 2). The first temporary gauge was operational from 9 December 1997 to 19 May 1998, and the second gauge was active between 21 January 1998 and 3 January 1999. The difference between mhhw and mllw at temporary gauge 1 was reported as 2.00 m. At temporary gauge 2 the difference was reported as 1.92 m. The tidal prism, calculated from current measurements taken in the inlet throat during 5-15 May 1975, was found to be 9.8×10^9 cu ft during spring tide (Florida Coastal Engineers, Inc., 1976).

At St. Marys Entrance, the mean wave height is 1.0 m, and the average period is 7.8 sec according to a long-term hindcast (http://frf.usace.army.mil/wis/). Most waves approach between the southeast to northeast. Winter waves are slightly larger than during other seasons. Sediment movement increases with the arrival of tropical and subtropical storms. The 10-year storm has a 2.1-m surge (Florida Coastal Engineers, Inc., 1976).

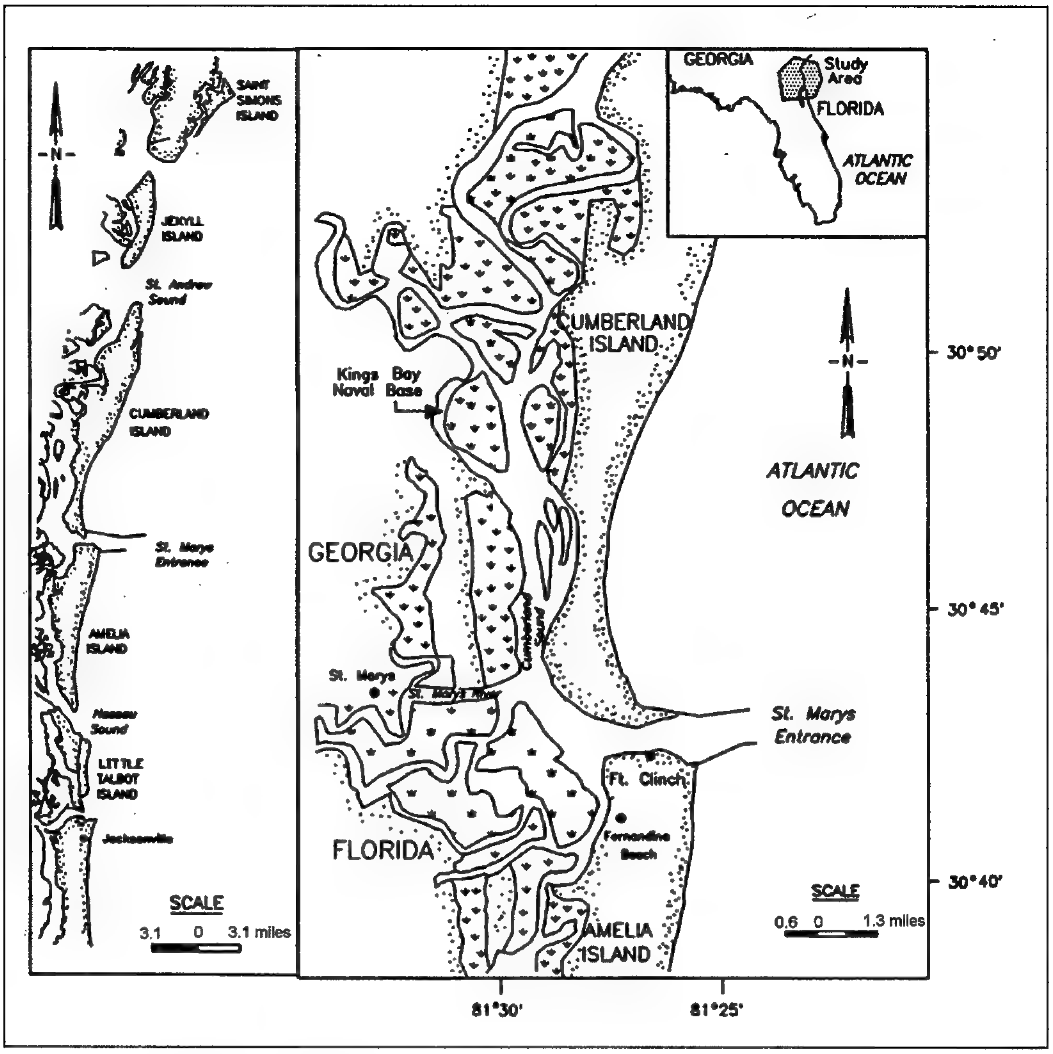

Figure 1. Location map for St. Marys Entrance, Florida (from Pope and Richardson 1994)

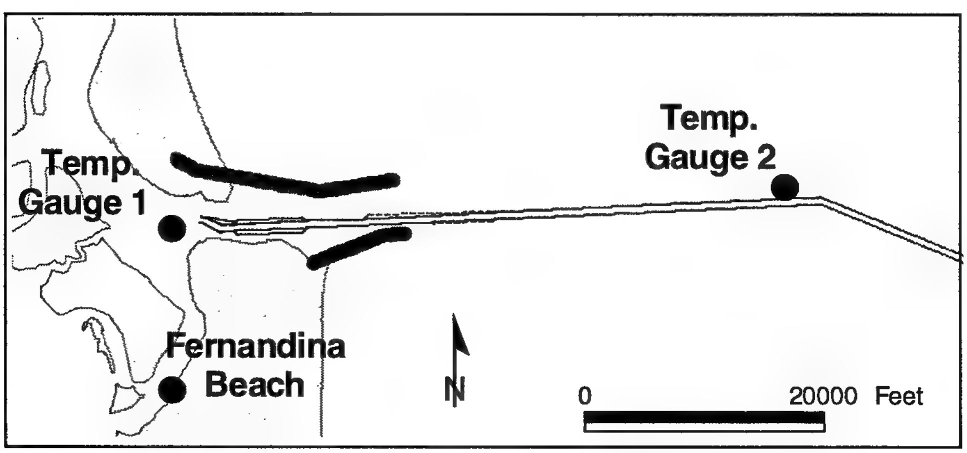

Figure 2. NOS tide stations in the vicinity of St. Marys Entrance

More than 200 sediment samples were collected from Amelia Island, Cumberland Island, and the St. Marys Entrance complex between July 1988 and May 1992 (Gorman, Pope, and Pitchford 1994). The sediment in this area was described as "well-sorted, fine to medium sands for Cumberland and Amelia Islands." The mean sediment size on Cumberland Island was 0.18 mm and on Amelia Island, 0.31 mm. On the northern fillet of St. Marys Entrance, the average grain size was 0.13-0.19 mm. The grain size on the southern fillet was 0.25 mm. Drilling logs of sediment samples provided by the Jacksonville District show that 40 percent of the samples taken from the channel consisted of medium to coarse sand.

Description of Navigation Project

In 1881, jetty construction began on both sides of the entrance, the first of several Federal engineering projects in this area. The jetties were completed in 1904. Originally, the jetty crest elevation was to be at mean low water (mlw) except for the outer 1,000 ft that was to be at midtide. Both jetties were raised to mean high water (mhw) by the River and Harbor Act of 1896 (U.S. Army Corps of Engineers (USACE) 1961 as cited in Gorman and Pope 1994). In 1926-1927, the north jetty was elevated to 6.9 ft mlw, and the south jetty was elevated to 5.9 ft mlw. Although the jetties were sand tightened in the 1980's, their configuration and dimensions have not changed since the 1920's (Figure 3). Since the 1920's, both jetties have settled, and the jetty crest elevation has decreased. Because the jetty crest is below mhw, water level is above the jetties through much of the tidal cycle (Figure 4–7). Water level in the surf zone on the adjacent beaches will rise above that measured by a tide gauge (such as at the aforementioned NOS temporary gauges) through wave-induced setup and the setup created by an onshore-directed wind. Water can flow both through and over the jetty, depending on the phase of tide and the wave conditions. Scour holes on the south side of the north jetty are created by water flowing through the structure and into the entrance (Figures 8-10).

Placement of the jetties modified the hydraulic characteristic of the inlet (Florida Coastal Engineers, Inc., 1976; Olsen 1977). The tidal flow through the inlet was directed to the center of the channel, the two historical channels across the ebb shoal were abandoned, and one new channel formed in a more central location. The new ebb jet constrained by the jetties moved the ebb shoal seaward at a rate of 1,300 ft/year.

There are two navigation projects within St. Marys Entrance, the Fernandina Harbor Navigation Project (FHNP) and the Naval Navigation Project (NNP) (Figure 11). The two channels are commonly referred to as the "civil channel" and the "military channel," respectively. The FHNP was authorized to maintain shipping access to the Port of Fernandina. Entry to the Kings Bay Naval Base in King Bays, GA, is provided by the NNP. Within the vicinity of the entrance the FHNP is completely within the NNP and requires no maintenance. The entrance channel is 500 ft wide at the bottom, except where there are channel wideners. In the vicinity of the wideners the channel width increases by 300 ft. The designed

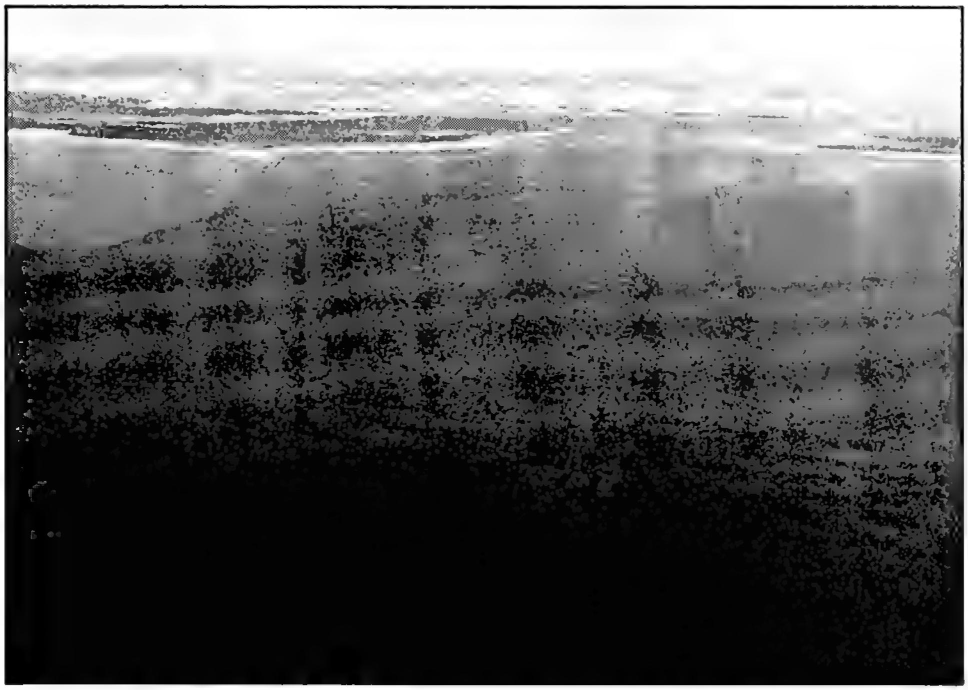

Figure 3. Jetties at St. Marys Entrance, looking west, 7 July 1993

Figure 4. South jetty looking southwest, at St. Marys Entrance, 28 February 2001

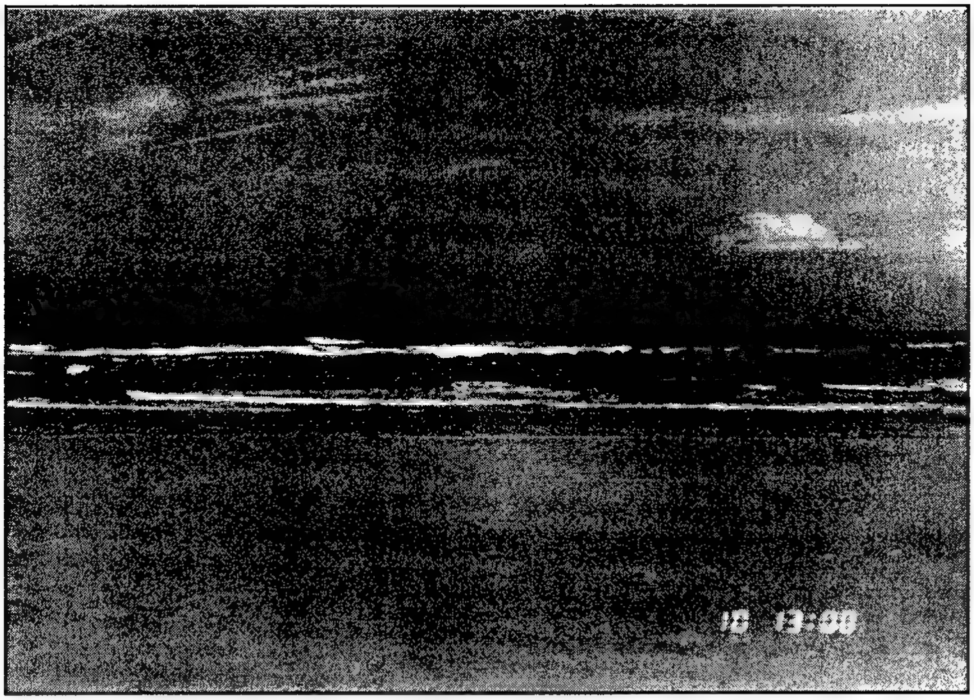

Figure 5. North jetty looking northeast, 11 October 1991

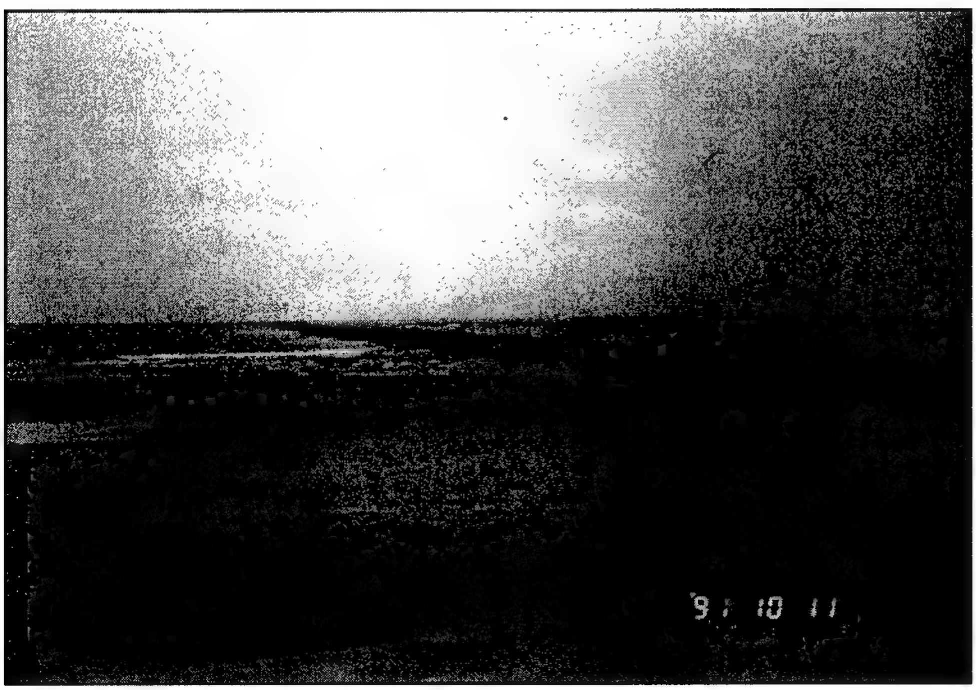

Figure 6. North jetty looking east, 11 October 1991

Figure 7. North jetty looking south, 11 October 1991

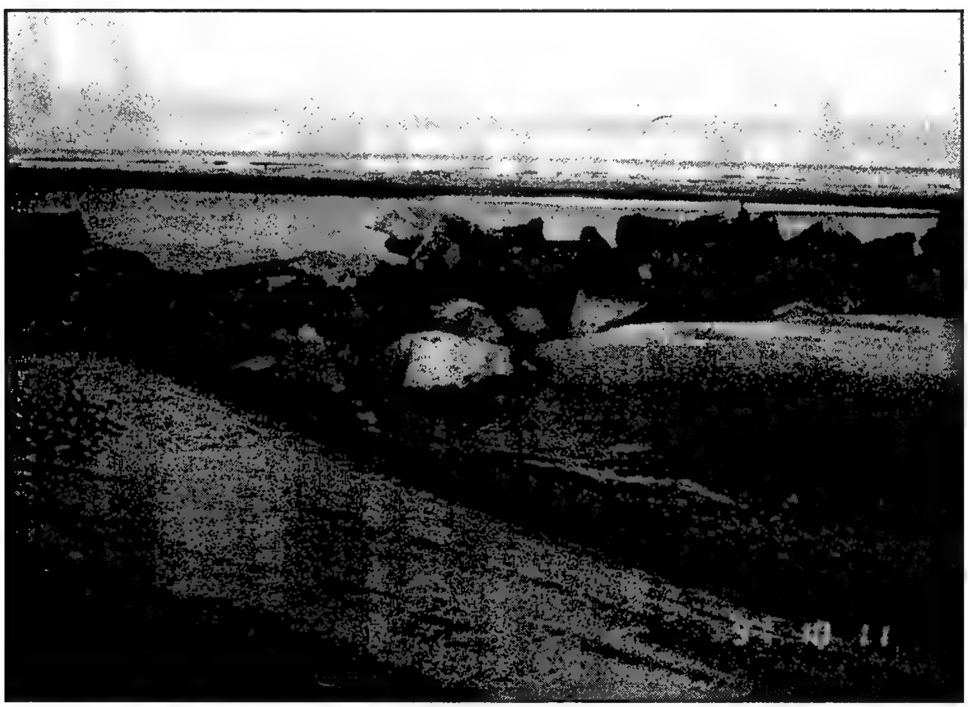

Figure 8. North jetty and scour in intertidal zone, 11 October 1991

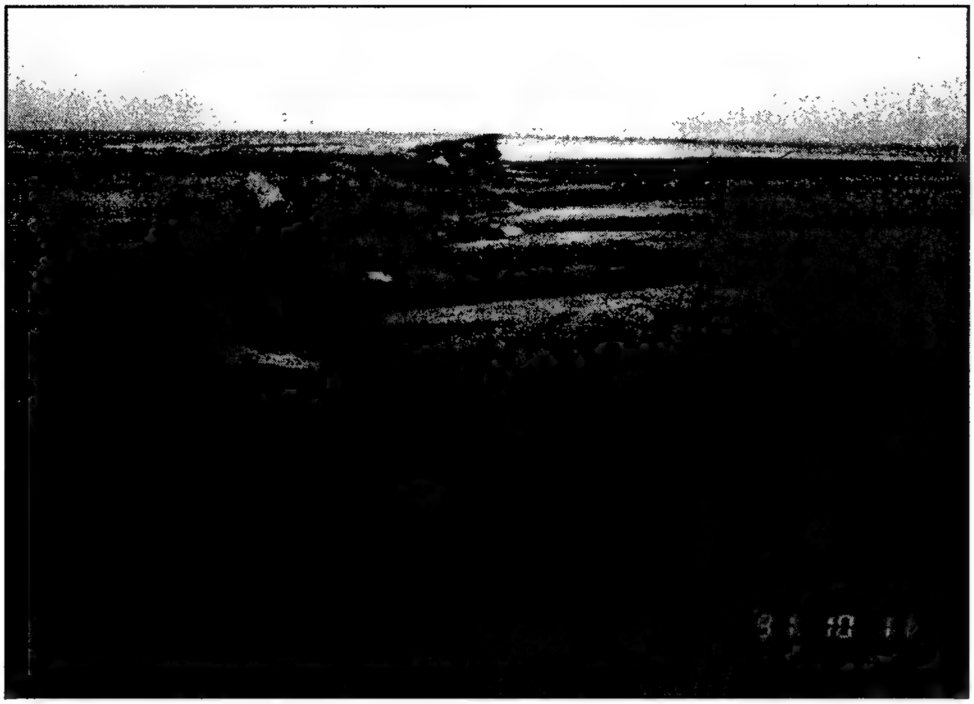

Figure 9. North jetty at low tide and scour holes in intertidal zone, 11 October 1991

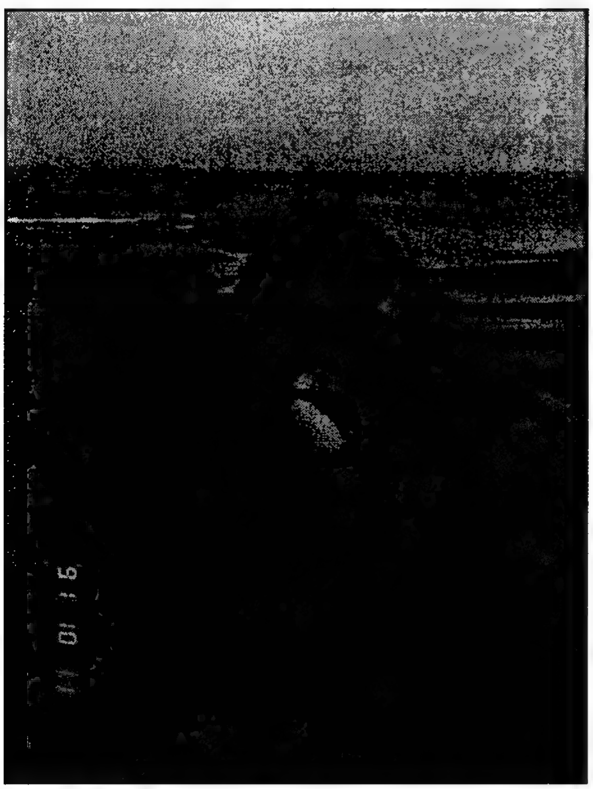

Figure 10. North jetty and scour holes on both sides, 10 October 1991

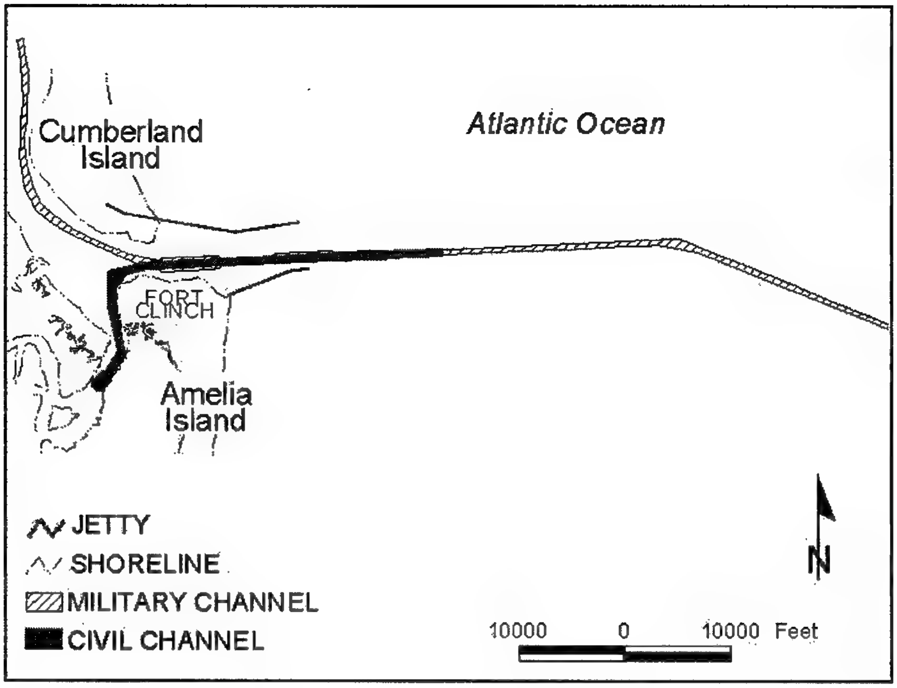

Figure 11. Federal Navigation Projects (after Raichle, Bodge, and Olsen 1997)

side slope of the channel is 3:1. In 1905, the channel was initially dredged to -19.1 ft mlw; in 1924, channel depth was increased to 27.9 ft mlw; in 1954, to 34.1 ft mlw; and in 1974, to 40.0 ft mlw. Presently, the channel is maintained at a depth of 51 ft mllw (46 ft authorized depth, 3 ft advance dredging, 2 ft allowable overdredging).[1] The channel extends 12.5 miles offshore to reach an ambient depth of 51 ft mllw.

Previous Studies on Longshore Transport

Sediment transported along the shore is a major supply to the inlet geomorphic complex and, therefore, a major contributor to channel shoaling. Along the southeastern coast of the United States, the regional net longshore sediment transport is directed to the south. The longshore transport rate varies in magnitude and may reverse locally, such as in the vicinity of an inlet.

There is geomorphic evidence that regional longshore transport in the vicinity of St. Marys Entrance is to the south. For example, the ebb shoal at St. Marys Entrance is offset to the south, extending 3 miles to the north and 6 miles to the south (Gorman 1991). The historic pattern of channel migration is another geomorphologic indicator of southerly longshore transport. Before the jetties were built, the material transported from the north would be deposited on Pelican Bank, north of the channel (Figure 12). The main ship channel would

[1] In 1997, the Jacksonville District changed its navigation vertical datum from mlw to mllw in conformance with practices instituted by the NOS (Headquarters, U.S. Army Corps of Engineers 1993). At St. Marys Entrance, the difference between the two datums is 0.2 ft. Because the authorized depth is rounded to the nearest foot, the datum conversion did not change the authorized channel depth.

then migrate to the south until it became hydraulically inefficient. Then the main channel would move north to Cumberland Channel as shown in Figure 12 (Olsen 1977). In a report on the Georgian coastline, Griffin and Henry (1984) supported their conclusion of southerly transport by documenting southern movement of coastal islands and the ebb shoal morphology.

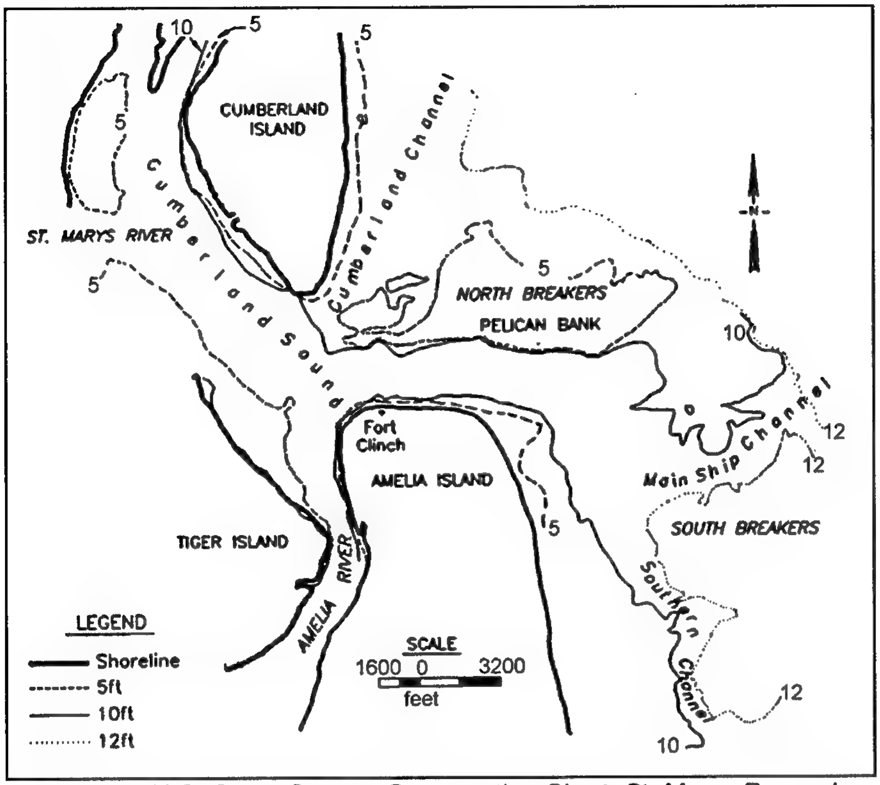

Figure 12. U.S. Coast Survey, Comparative Chart, St. Marys Bar and Fernandina Harbor, Florida, 1856 (Gorman and Pope 1994)

Modern estimates of longshore transport rates vary in magnitude and direction (Table 1). Reported net rates range from 90,000 to 600,000 cu yd/year. Some qualitative studies based on the changes in the local morphology suggest transport may be to the north along Amelia Island (Florida Coastal Engineers, Inc., 1976; Olsen 1977). Variations in longshore transport estimates can be explained if the method of determining the rate is understood.

Florida Coastal Engineers, Inc. (1976), calculated the longshore transport rate from a computerized 100-year sand budget. Richards and Clausner (1988) analyzed wave statistics from the Wave Information Study (WIS) (1956-1975) to estimate the average transport rates over an 11.5-mile cell centered 0.8 mile north of St. Marys Entrance. Modifications to the transport rate by the jetties are not evident in this value because it is averaged over a distance. The Kings Bay Coastal and Estuarine Physical Monitoring and Evaluation Program (Grosskopf and Kraus 1994) calculated the longshore transport with the same WIS data set, but with a finer grid cell size (300 ft). Calculations with the smaller cell size indicated more variation along Cumberland Island and Amelia Island. The net transport adjacent to the inlet along both islands was found to be directed toward the inlet by Olsen (1977) and Grosskopf and Kraus (1994).

Table 1
Annual Sediment Transport Rate Estimates for St. Marys Entrance, cu yd/year

Source	South	North	Net	Gross
University of Florida as cited in Richards and Clausner (1988) and Parchure (1982) ; no documentation on method	380,000	142,000	238,000 S	522,000
Florida Coastal Engineers, Inc. (1976), p 44			500,000	
Richards and Clausner (1988)	544,000	454,000	90,000 S	1,008,000
Richards and Clausner (1988)			90,000 S	1,000,000
Dean (1988)	600,000	0	600,000 S	600,000
Grosskopf and Kraus (1994)	Amelia Island: 26,000 Cumberland Island: 116,000	Amelia Island: 133,000 Cumberland Island: 1,300	Amelia Island: 106,000 N Cumberland Island: 114,000 S	Amelia Island: 161,000 Cumberland Island: 119,000
USACE (as cited in Parchure 1982)	600,000	100,000	500,000 S	700,000
Pope (1991)	Southerly transport based on ebb shoal morphology, not quantified			
Raichle, Bodge, and Olsen (1997), p13	Southerly transport except for localized reversal within 1,000 ft of the inlet			

Purpose and Structure of Study

The objective of this study is to determine the causes of shoaling at St. Marys Entrance and to arrive at alternatives to ongoing maintenance practices that will reduce the frequency and cost of dredging, while maintaining reliable navigation. The objective is met through the application of the DMS concepts and methodology. The following steps were followed:

a. Inspect the site and critically review relevant literature.

b. Develop a Geographic Information System (GIS) database within the DMS Data Manager. Describe shoaling conditions qualitatively and quantitatively based on the information within the Data Manager.

c. Analyze dredging records for the study site.

d. Establish and calibrate a numerical hydrodynamic model for the depth-averaged, two-dimensional circulation and run model with the existing conditions and proposed alternatives to infer sediment pathways.

e. Hindcast the nearshore wave climate to understand directions of wave-induced longshore transport.

f. Synthesize information and results from steps *a-e*, and present recommendations for reduction of channel shoaling.

This report documents the results of the DMS application to the performance of the St. Marys Entrance Channel. Specifically, it addresses the function of the channel widener (Navy Project or military channel). Based on information generated in this study, recommendations for the maintenance of the Fort Clinch rehabilitated groin field are also presented. Chapter 2 reviews the dredging records, from which channel shoaling patterns are identified and quantified. The procedures and results of the wave and current numerical modeling are presented in Chapters 3 and 4, respectively. Chapter 5 reviews the existing conditions and presents proposed alternatives. The conclusions and recommendations of this study are given in Chapter 6.

2 Dredging and Shoaling History and Analysis

Maintenance of St. Marys Entrance is necessary to assure reliable access to the Port of Fernandina and to the Kings Bay Naval Base. The Entrance has been maintained by dredging in 31 years since 1955. The channel has been dredged every year but three since 1987, when it was deepened to accommodate the Trident submarines. Annual maintenance of the entrance was about $3.5 million during the 1990's. This chapter discusses past and recent dredging practice, and shoaling trends are quantified from channel surveys.

Dredging History

Dredging records for this analysis were provided by the Jacksonville District. Records of dredging were available starting from 1980 (Appendix A). Between 1955 and 1980, individual dredging records were not available, but the Jacksonville District provided the dredging dates, dredged volumes, and cost of the dredging operations for that time period.

Since 1955, 28,895,000 cu yd of sediment has been dredged from the St. Marys Entrance (Figure 13). Of this volume, 11,912,000 cu yd was for maintenance, and 16,983,000 cu yd was new work. As the dimensions of the channel have increased, the volume of maintenance dredging has also increased. For example, after the channel was last expanded in 1987, the average maintenance volume went from 230,000 to 818,000 cu yd/year. The channel expansion in 1987 increased the channel width by 98 ft and the channel depth by 11 ft. With the western 9 miles of the channel requiring maintenance by the Jacksonville District, the additional volume of channel maintenance brought by the channel expansion was 1,897,000 cu yd (11 ft x 98 ft x 9 miles). This volume of additional maintenance illustrates that the increase in maintenance after 1987 was caused mainly by channel expansion rather than by a change in environmental conditions.

Reliable channel survey station locations are available for determining the exact area of dredging from 1986 on. Since that time, 11,780,000 cu yd has been dredged from the entrance for maintenance. Most of the dredging was performed between sta 100 and 340 (Figures 14 and 15). A volume of 10,722,000 cu yd has been removed from this area, whereas only 1,021,000 cu yd has been dredged from the remaining 260 stations.

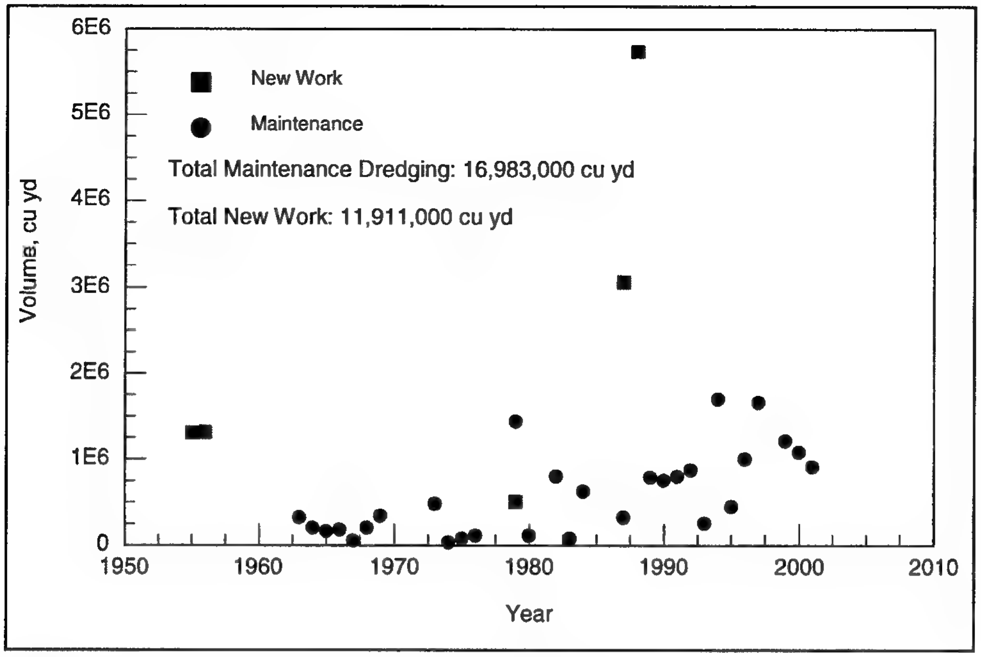

Figure 13. Volume of material dredged since 1955

Until 1979, all dredged material was either deposited offshore or side-cast within the channel (Figure 16). Since 1979, 18 percent of the dredged material has been placed on Amelia Island and 14 percent has been placed in the nearshore zone off Amelia Island. The remaining 68 percent was disposed offshore (Table 2).

The placement of dredged material based on sediment type was analyzed using sediment type and disposal location determined from the dredging records. Events with no station location were omitted. The results indicate that effectively all of the dredged material placed on the beach was sand. Ocean disposal of dredged material was 7 percent sand, 79 percent silt, 4 percent was silt/clay, and 10 percent sand/silt. Fifty-three percent of the sand removed from the channel was placed on the beach, and 93 percent of the silt went to the ocean.

Channel Shoaling

Shoaling rates have been documented by Smith, Pope, and Gorman (1994) and Raichle, Bodge, and Olsen (1997). The shoaling analysis presented in this section reviews previous conclusions and presents recent shoaling data (1997-2000) based on bathymetry surveys of the channel provided by the Jacksonville District.

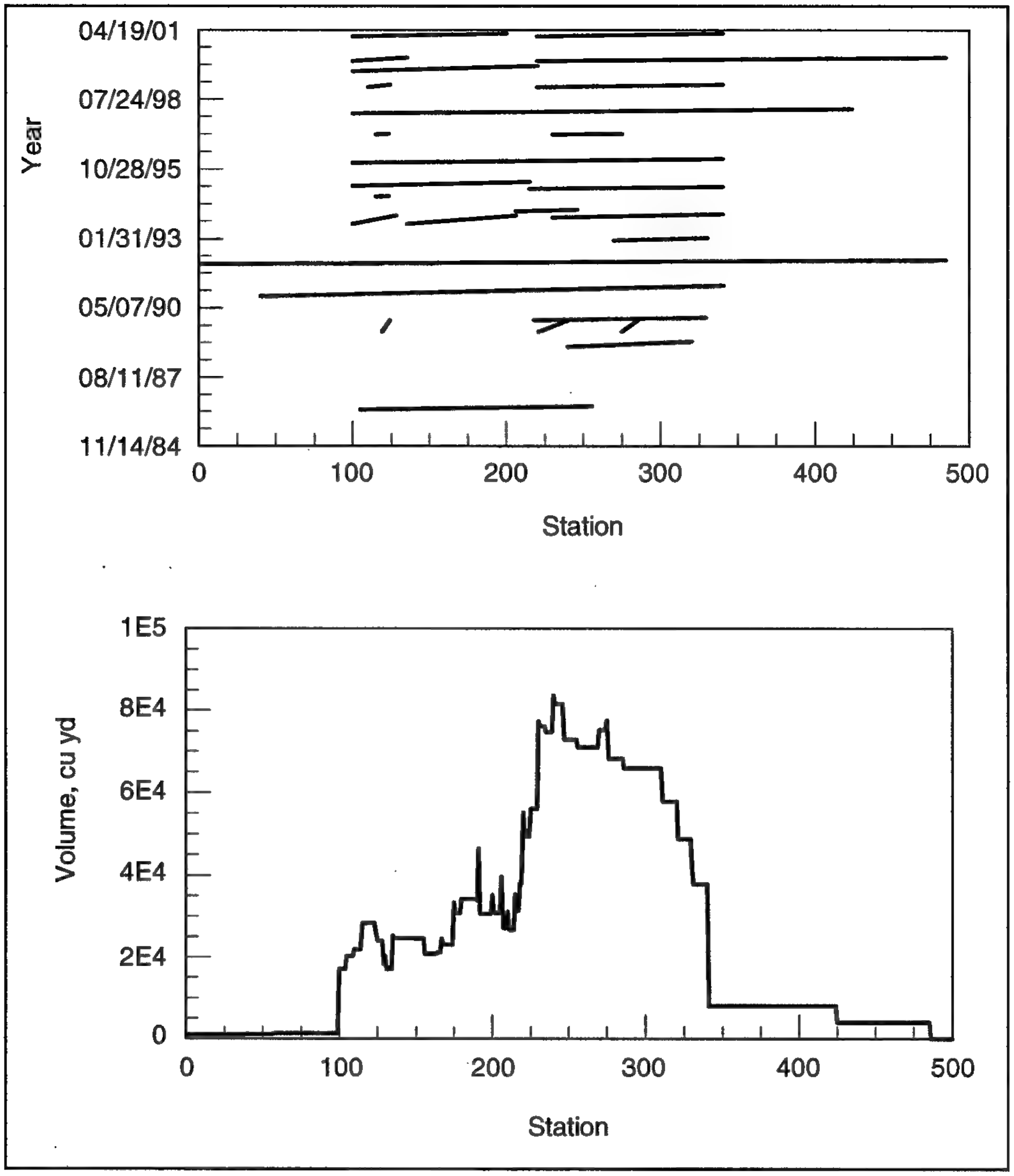

Figure 14. Dredging frequency and volume by station, 1986-2001

Causes of shoaling

Four potential causes of shoaling in the St. Marys Entrance channel were identified in this study (Figure 17):

a. Introduction of sediment by longshore transport, from either the north or the south.

b. Migration of bed forms into the channel.

c. Deposition of fine sediment along the portion of the channel located seaward of the ebb shoal.

d. Episodic shoaling associated with larger storms.

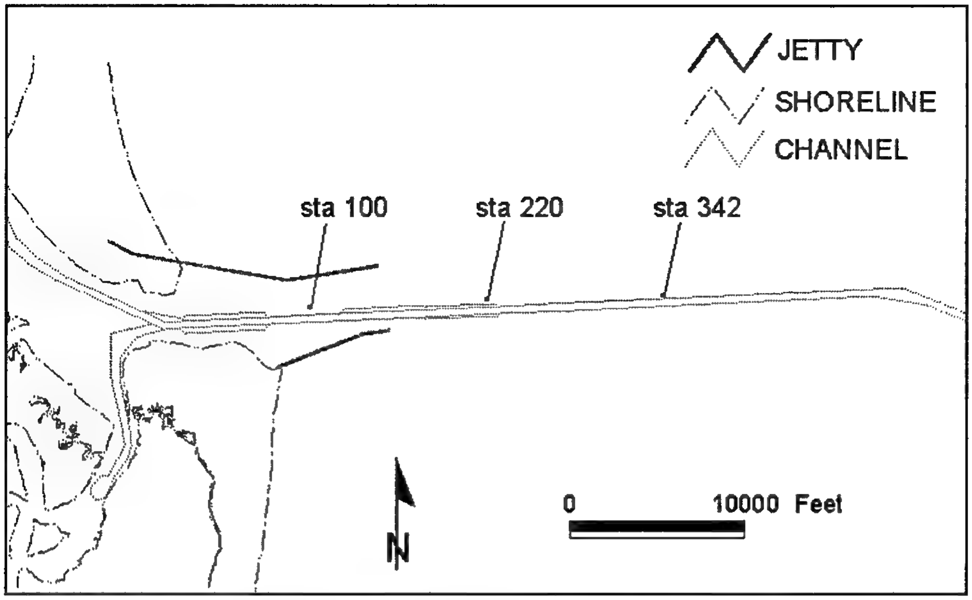

Figure 15. Channel diagram showing stations where majority of dredging has been conducted since 1986

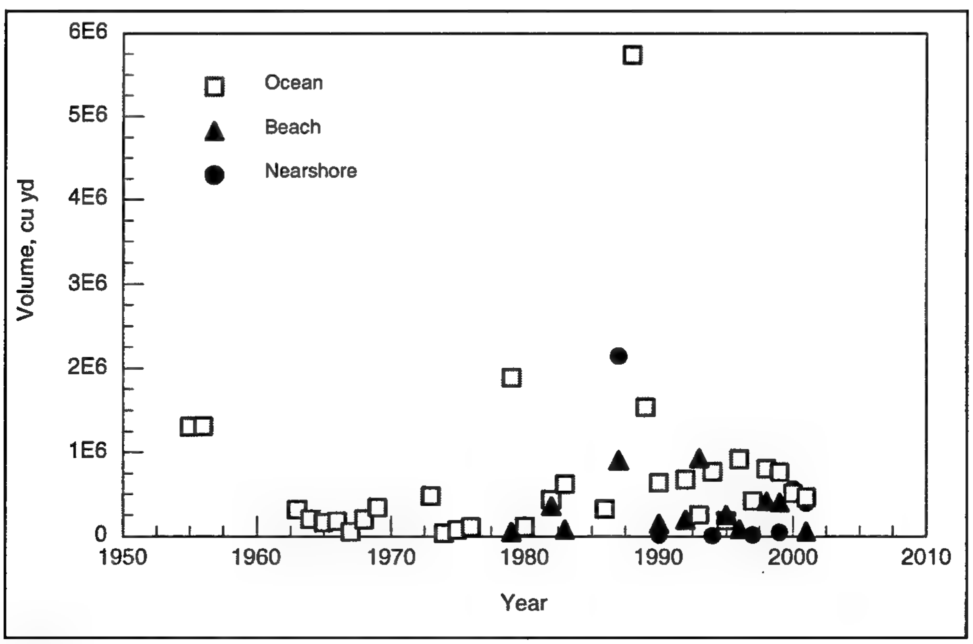

Figure 16. Historical locations of dredged material placement

Table 2 Dredged Sediment Disposal Locations Since 1980		
Disposal Area	**Volume, cu yd**	**Percent**
Beach	3,826,487	17
Nearshore	3,184,567	14
Ocean	15,174,812	68
Total	22,185,865	100
Note: Numbers listed are summations from dredging records and do not imply precision to the number of figures presented.		

Several publications have presented shoaling rates for St. Marys Entrance (Table 3). The results were originally reported in either non-SI or SI units, depending on the source. In Table 3, the shoaling rates have been converted to cu yd/year. If a unit of length was given, the units were converted to cu yd/year/200 ft.[1] The shoaling rate for the entire channel ranges from 106,000 to 1,350,000 cu yd/year, a range varying by an order of magnitude. However, the larger estimate is a maximum cumulative estimate, and the smaller estimate is based on the dredging records from 1954 to 1973. From 1954 to 1973, the channel was narrower and shallower than it is now so there would have been less dredging requirement. It is also possible that documentation of the dredging during that time is now incomplete. The shoaling rate has increased over time from 106,000 cu yd/year in the 1950's and 1960's to 806,000 cu yd/year in the late 1980's and early 1990's (Smith et al. 1994). The increase is due most likely to the increasing authorized or required channel depth after deepening in 1974 and 1987.

Discussion of the shoaling patterns at St. Marys Entrance can be organized by location along the channel. The causes and characteristics of shoaling are different in the inner and outer channels. The inner channel is controlled by sediment supplied by longshore transport. The DMS methodology (Kraus et al., in preparation) attributes shoaling in this environment to the vertical expansion of the channel as the longshore current flows perpendicular to the project channel (Figure 18). In shallow water adjacent to the channel, the current flowing perpendicular to the channel transports sediment toward the channel. Where the sediment-laden longshore current experiences the greater depths (and thus, weaker velocities) in the channel, the sediment is deposited. Shoaling in the outer channel is caused by vertical and horizontal expansion of the channel (Kraus et al., in preparation). Vertical expansion occurs as the channel moves off the ebb shoal and horizontal expansion occurs as the channel moves past the

[1] All along-channel distances have been normalized to 200 ft. This distance was chosen because the shoaling rates calculated in this report were calculated over 200 ft rather than 100 ft to decrease calculation time.

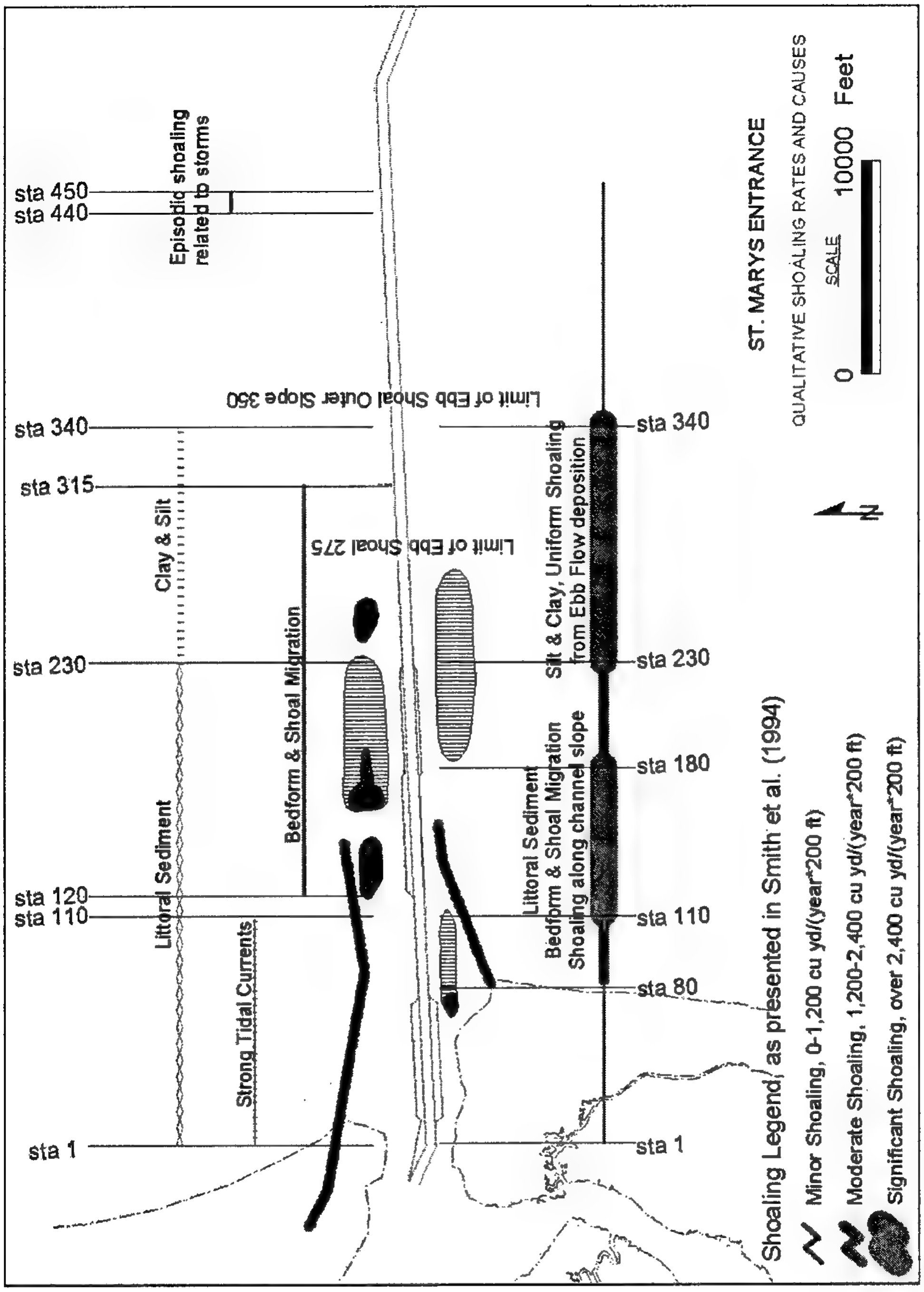

Figure 17. Identified causes and rates of shoaling in the entrance channel

Table 3
Estimated Shoaling Rates

Reference	Shoaling Rate[1]
ESI 1986[2] (as cited in Smith, Pope, and Gorman (1994)	1,350,000 cu yd/year
Vemulakonda et al. (1988)	602,500 cu yd/year, sta 77-481
U.S. Army Engineer District, Jacksonville (1993)	16,100 cu yd/(year*200 ft), sta 226-331, Dec 89-Jun 90
	20,000 cu yd/(year*200 ft), sta 275-286, Dec 89-Jun 90
	17,100 cu yd/(year*200 ft), sta 290-302, Dec 89-Jun 90
	1,200 cu yd/(year*200 ft), sta 345-375, Jun 88-Jun 90
	700 cu yd/(year*200 ft), sta 350-360, Jun 88-Jun 90
Smith et al. (1994)	806,000 cu yd/year, 1988-1992
	355,800 cu yd/year, 1974-1987
	105,900 cu yd/year, 1954-1973
	2,500 cu yd/(year*200 ft), sta 110-180, Feb 88-Mar 91

[1] All rates converted to cu yd/year/200 ft for comparison to shoaling rates calculated in this report. Rates were rounded to the nearest hundred.
[2] Kings Bay Environmental Study. (1986). "Final third supplement to the Environmental Impact Statement for preferred alternative location for a Fleet Ballistic Missile Submarine Support Base, Kings Bay, Georgia (St. Marys Entrance)," unpublished report, Department of Navy, Officer in Change of Construction, Trident, St. Marys, Georgia.

confines of the jetties. Wave-induced longshore transport interrupted by the inlet is directed seaward by the ebb tide. Sediment settles out in the region of relative calm away from the strong currents near the inlet mouth. Near the edge of the ebb shoal, sediment is also supplied by ebb current that runs along the axis of the channel. The velocity decreases because of the horizontal spreading of the ebb jet and depth-dependent vertical increase in water depth (Figure 19).

Inner channel shoaling

Longshore transport can supply sediment to the channel from the north and from the south, and it is likely the predominant sediment source for shoaling landward of sta 230. Figures 20-22 show the change in seafloor elevation at St. Marys Entrance. Typically, the change is less than 1 ft. The only area that consistently shoals more than 4 ft is located near channel marker R-22 or sta 120. This shoal will be discussed in detail in the following paragraphs.

Regionally, longshore transport is directed from north to south; locally, however, sand enters the inlet laterally from both directions. Net northward transport from the northern end of Amelia Island is possible for four reasons. First, the ebb shoal shelters this area from large northeasterly waves, reducing southward transport. Second, there is a summer reversal in the wave direction, with summer waves typically incident from the southeast. These waves are smaller and do not carry as much material as typical winter waves out of the

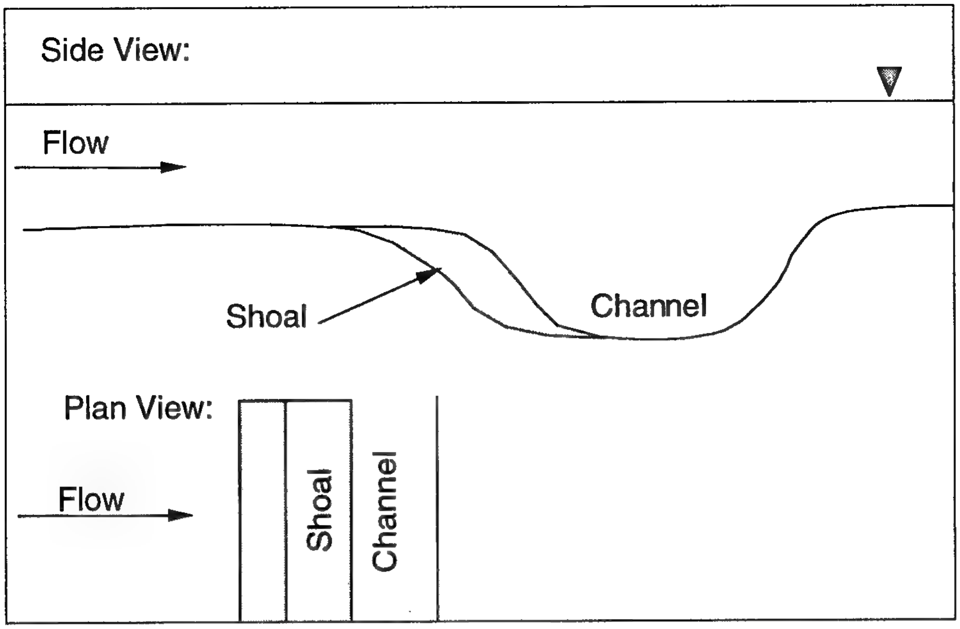

Figure 18. Shoaling due to vertical expansion and cross-channel flow from the DMS Manual

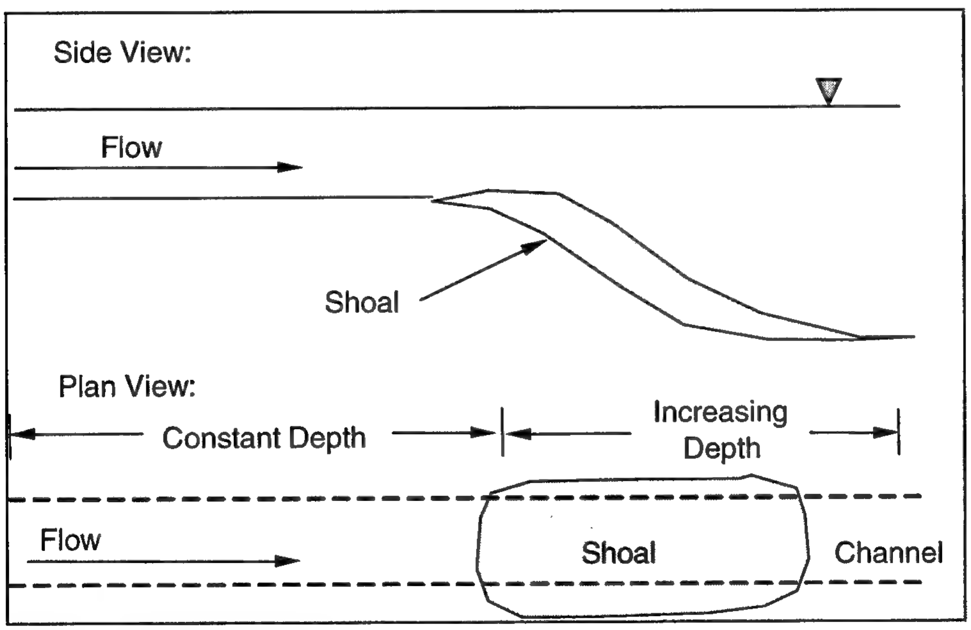

Figure 19. Shoaling due to vertical channel expansion, in-line channel flow from the DMS Manual

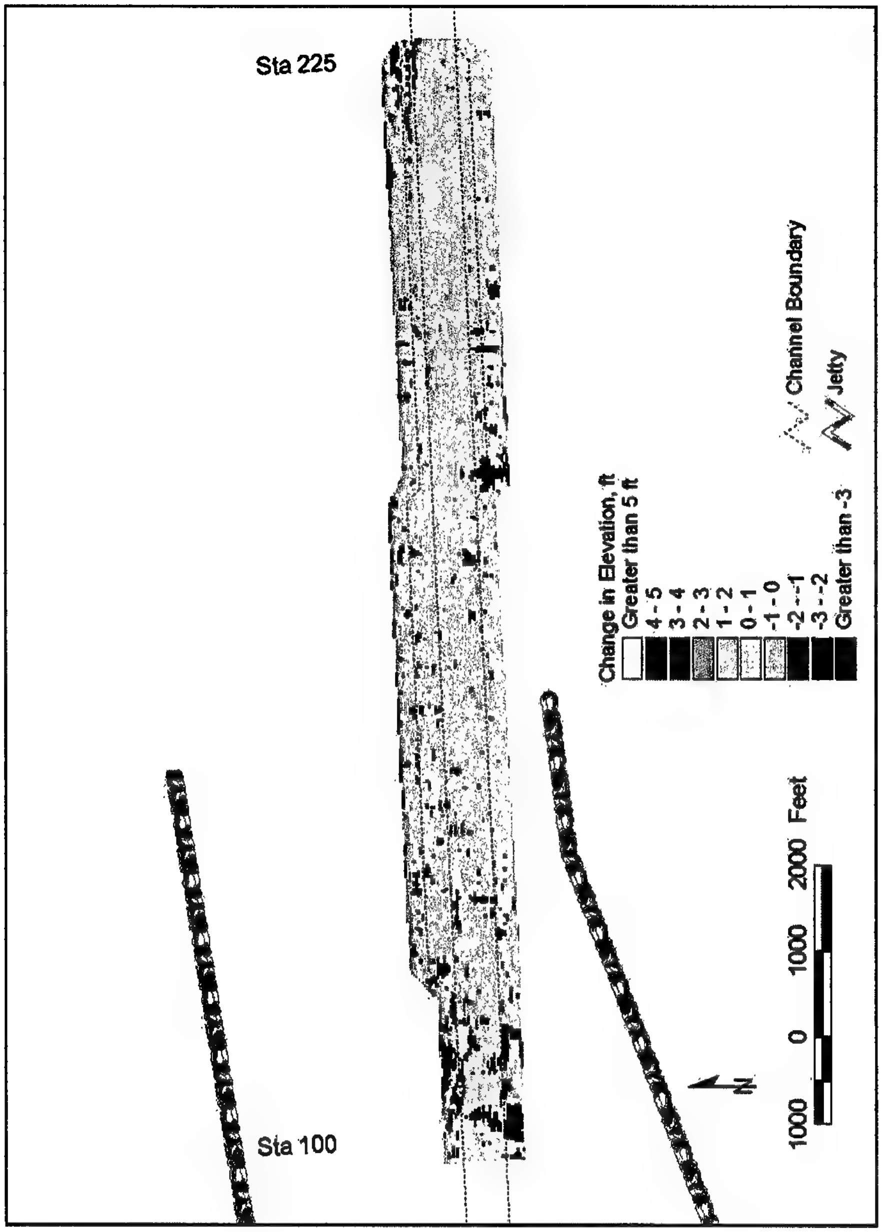

Figure 20. Elevation change between February and November 2000

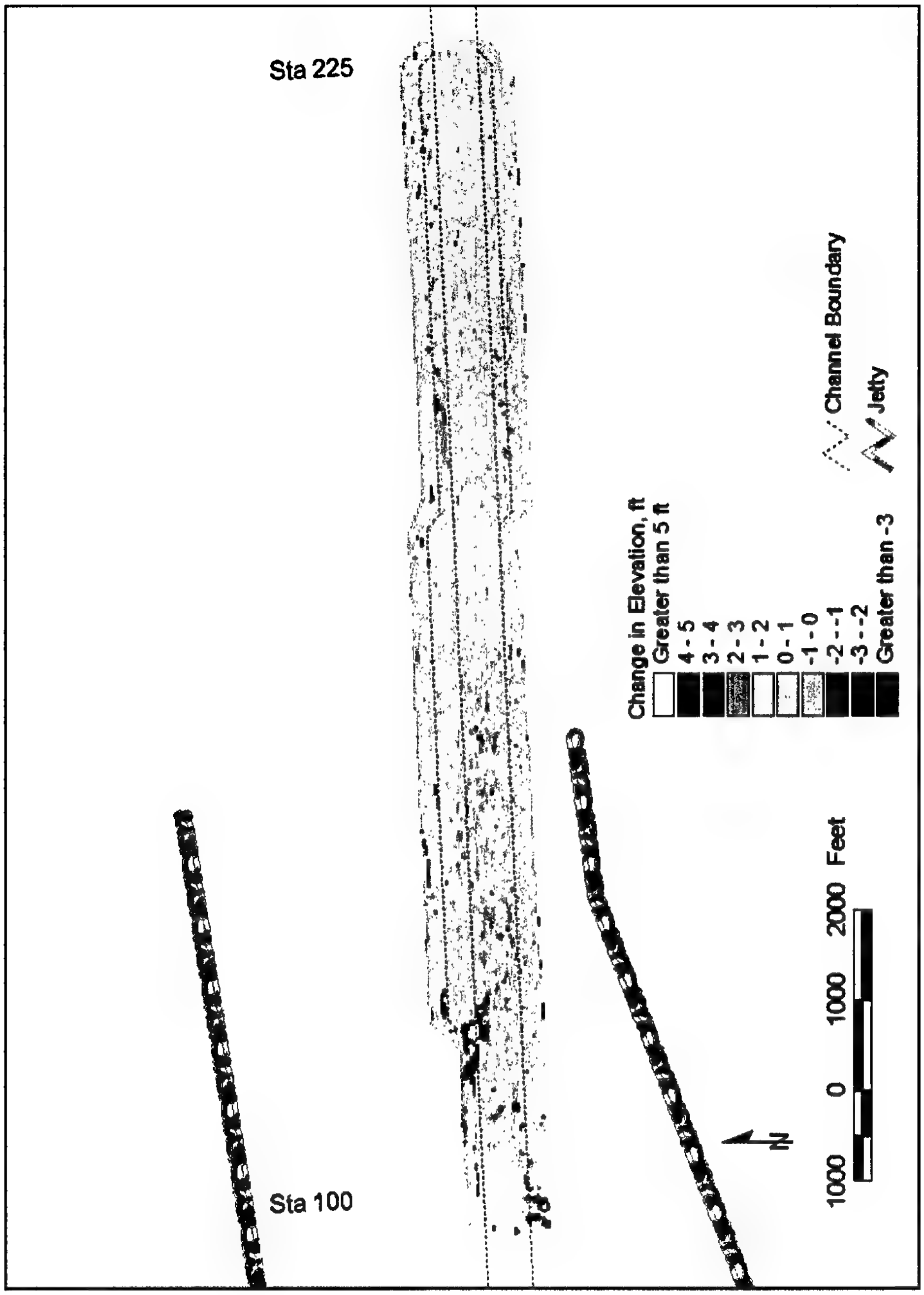

Figure 21. Elevation change between March and July 1997

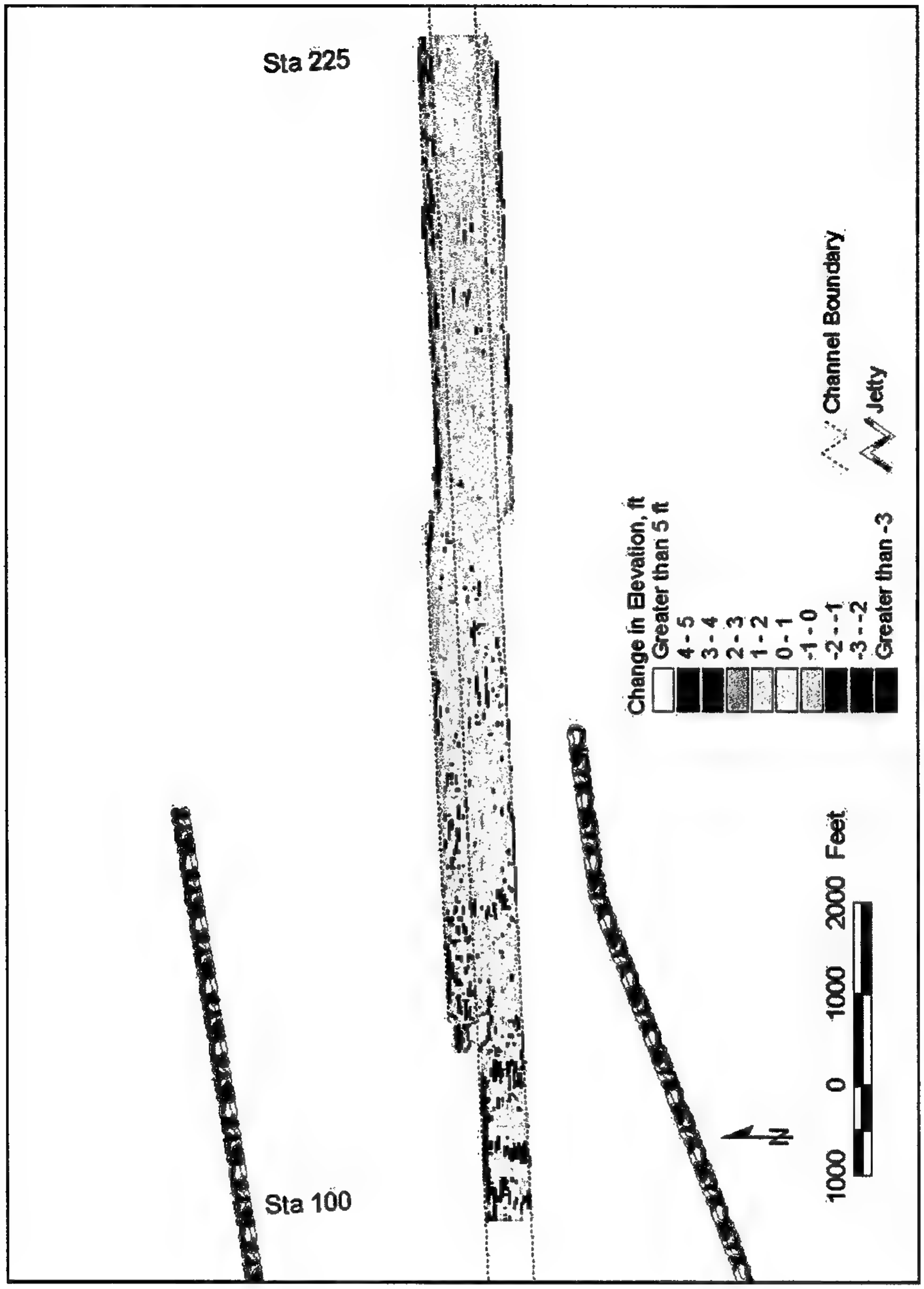

Figure 22. Elevation change between April 1996 and March 1997

northeast, but still supply sand to the inlet for several months. Third, waves approaching from the east, the dominant wave direction throughout the year, refract around the ebb shoal and travel northward as they approach the beach. Sand carried along shore can enter the inlet through the permeable jetties that are submerged during higher tidal levels. In May 1975, it was estimated that as much as 28 percent of the tidal current would flow through the jetties (Florida Coastal Engineers, Inc., 1976; Olsen 1977). Since then, the landward 1,500 ft of the south jetty has been sand tightened so the flow through that jetty has decreased. This has reduced the amount of sand available to the interior shoreline of Amelia Island (Fort Clinch) (Raichle, Bodge, and Olsen 1997). The fourth potential mechanism for northward transport is a clockwise rotating eddy south of the channel during the ebb tide.

Sta 1-90 experience little or no shoaling within the channel. A spit extending from Cumberland Island approaches the channel from the north, but does not extend into the channel, probably because strong currents scour this area (velocities on the order of 0.8 m/sec according to calculations presented in Chapter 4). The ebb current remains constricted between Cumberland and Amelia Island until sta 90, which is parallel to the Amelia Island shoreline. Here, the entrance widens, and the tidal current decreases. From sta 90 seaward, the ebb current is no longer strong enough to remove sediment from the channel.

The stretch between sta 90 and 120 is a transition from minor shoaling to the west to severe shoaling to the east. Longshore transport supplies sediment to the channel from the north and the south, and the decreasing tidal current does not have sufficient strength to remove it.

Shoals and bed forms have been identified outside of the channel. These features can migrate into the channel and limit navigable depth. North of the channel, bed forms have been found from sta 160 to 230. The bed forms described here are less than 5 ft above ambient depth. Various types of bed forms have been identified between sta 190 and 230. There is no dominant bed form, suggesting this area has an active and dynamic sediment transport environment (Aubrey, McSherry, and Spencer 1991). Sta 160-190 are dominated by sand waves (termed sand ridges by the authors) that are 1 to 3 ft long. South of the channel, bed forms can be found from sta 72 to 110 and from sta 180 to 272 (Aubrey, McSherry, and Spencer 1991). South of the channel, bed forms longer than 12 ft can be found. Between sta 189 and 272 there are also shorter sand waves, less than 3 ft long. Smaller bed forms have been identified within the channel. These bed forms are 3-12 ft long and are elevated less than 5 ft above ambient depth, most only 2 to 3 ft high (Aubrey, McSherry, and Spencer 1991).

An area of critical shoaling has been identified by the Jacksonville District and through bathymetric surveys near channel marker R-22, sta 120. This area exhibits the highest shoaling rate within the inner channel. Figures 23 and 24 show this shoal, defined by the -45-ft mllw contour, encroaching on the channel.

Although most bathymetric surveys have been limited to the navigation channel, a survey was made in 1979 that covered the entire area between the jetties. In the 1979 survey, sand waves are apparent to the north of the channel (Figure 25). Woods Hole Oceanographic Institution monitored St. Marys

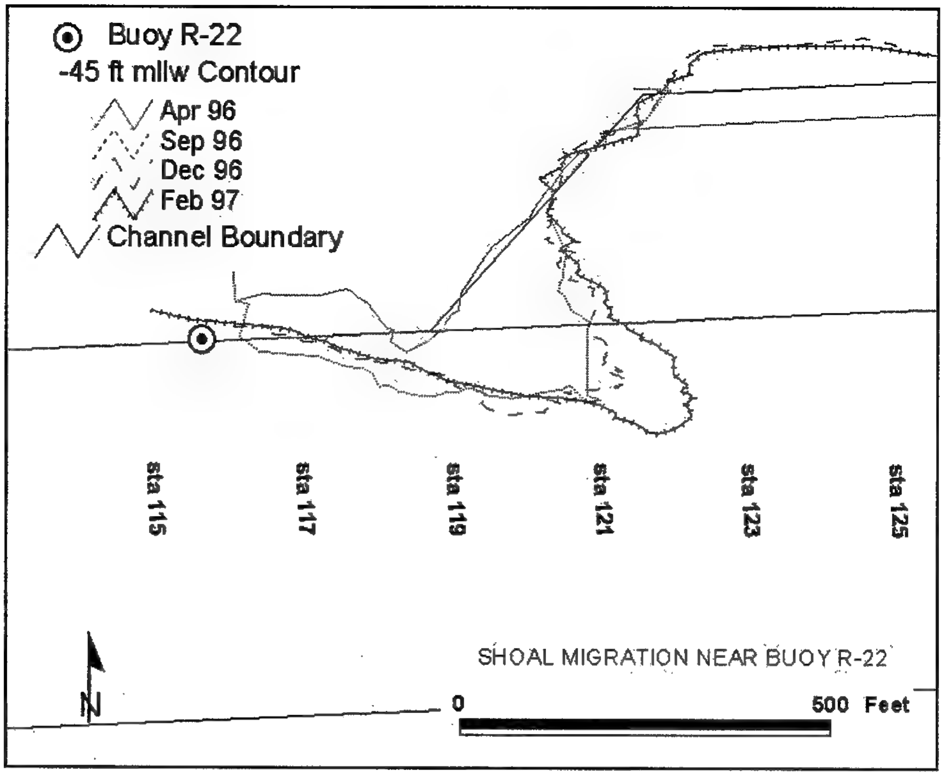

Figure 23. Shoal migration near buoy R-22, near sta 120

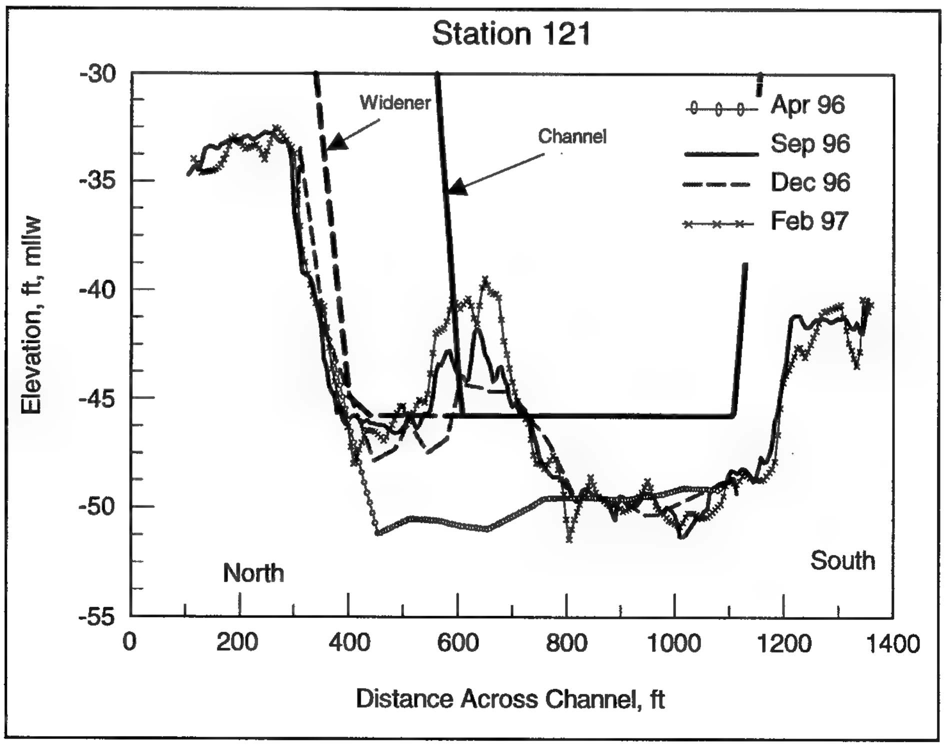

Figure 24. Channel cross section at sta 121

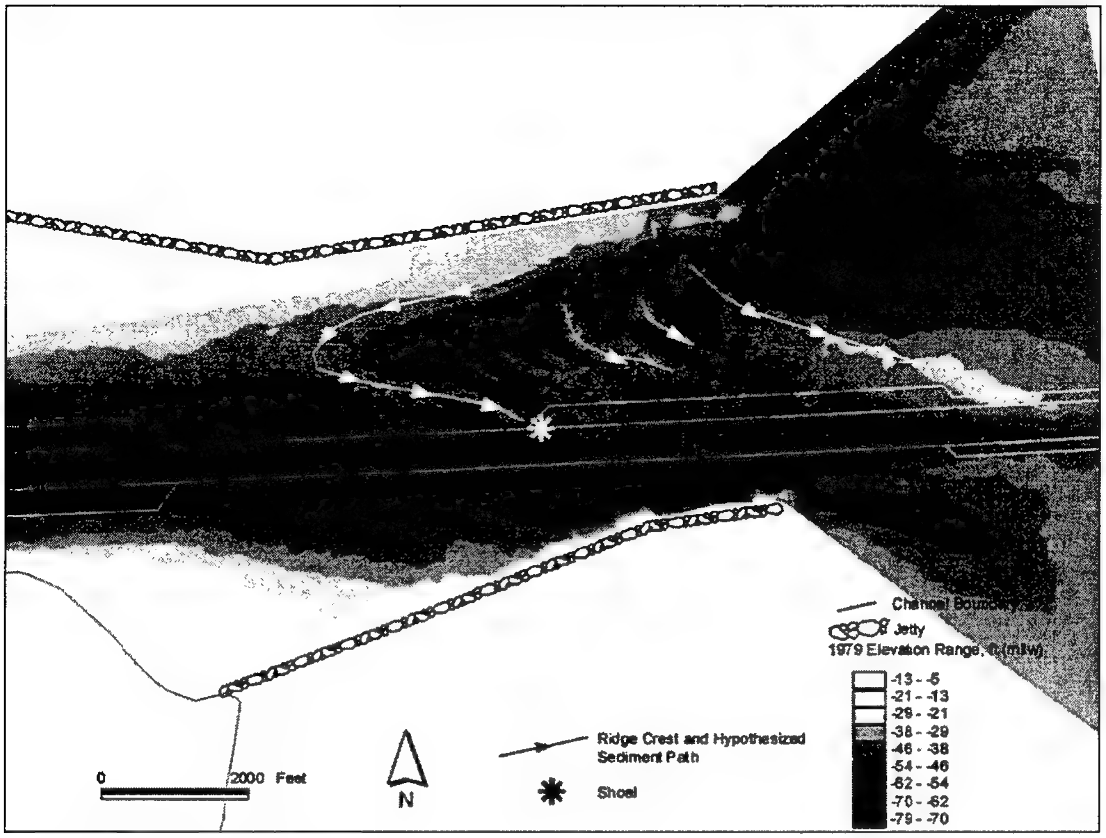

Figure 25. Bathymetry of 1979

Entrance from 1988 to 1990 (Aubrey, McSherry, and Spencer 1991). They also reported sand waves between sta 120 and 150. The westernmost wave is in line with the shoal that migrated into the channel between 1996 and 1997, indicating that this feature has persisted for more than 20 years. In 1987, channel wideners were added to the authorized channel through this reach to decrease the frequency of dredging by delaying shoal migration into the navigation channel.

Outer channel shoaling

Seaward of sta 180, channel shoaling of sediment supplied by longshore transport begins to taper, and shoaling is minor until sta 230. As the ebb jet flows past the ebb shoal, it weakens and deposits finer sediment, causing significant shoaling from sta 230 to 340 (Figures 26-28). Shoaling is more severe in the outer channel than in the inner channel.

The ebb tidal current loses speed as it flows into deeper water, and fine sediment deposits on the seafloor. The ebb current slows from 0.6 to 0.2 m/sec between sta 200 and 350 as the ebb jet extends beyond the ebb shoal and into deeper water (Figure 29). Calculation of the ebb current velocity is described in Chapter 4. Shoaling seaward of sta 275 is typically uniform over the width of the

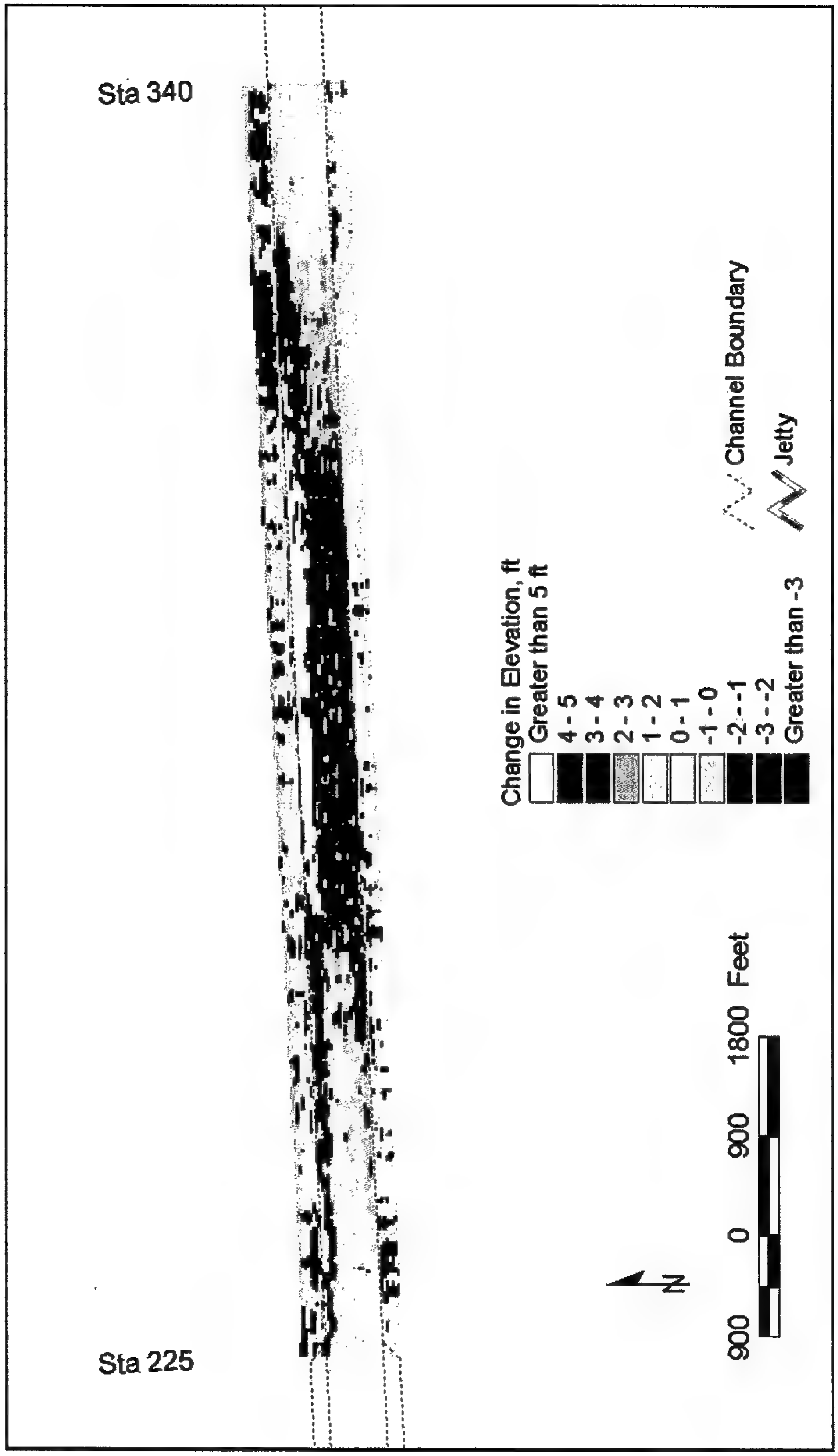

Figure 26. Elevation change between February and November 2000

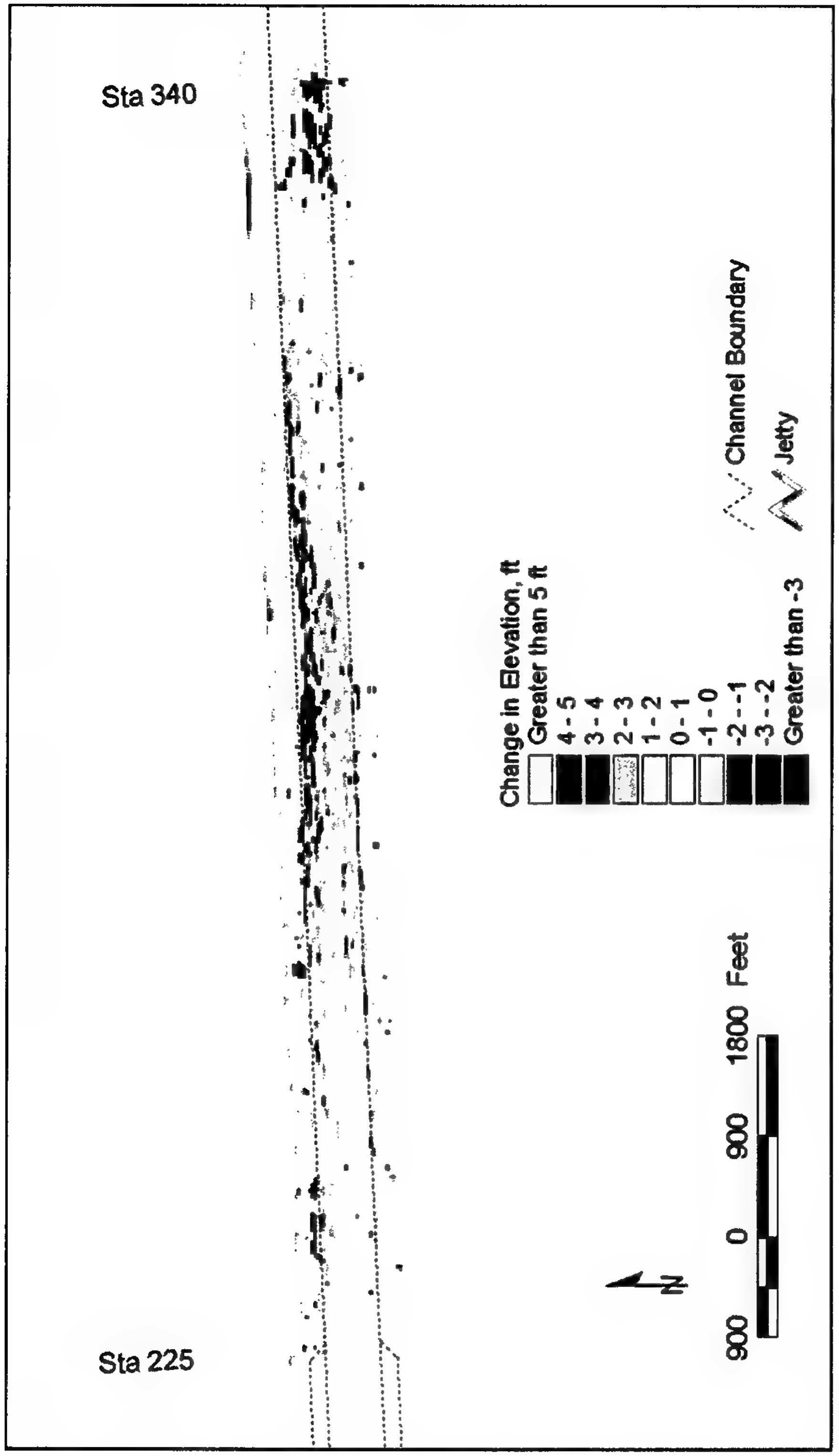

Figure 27. Elevation change between March 1996 and July 1997

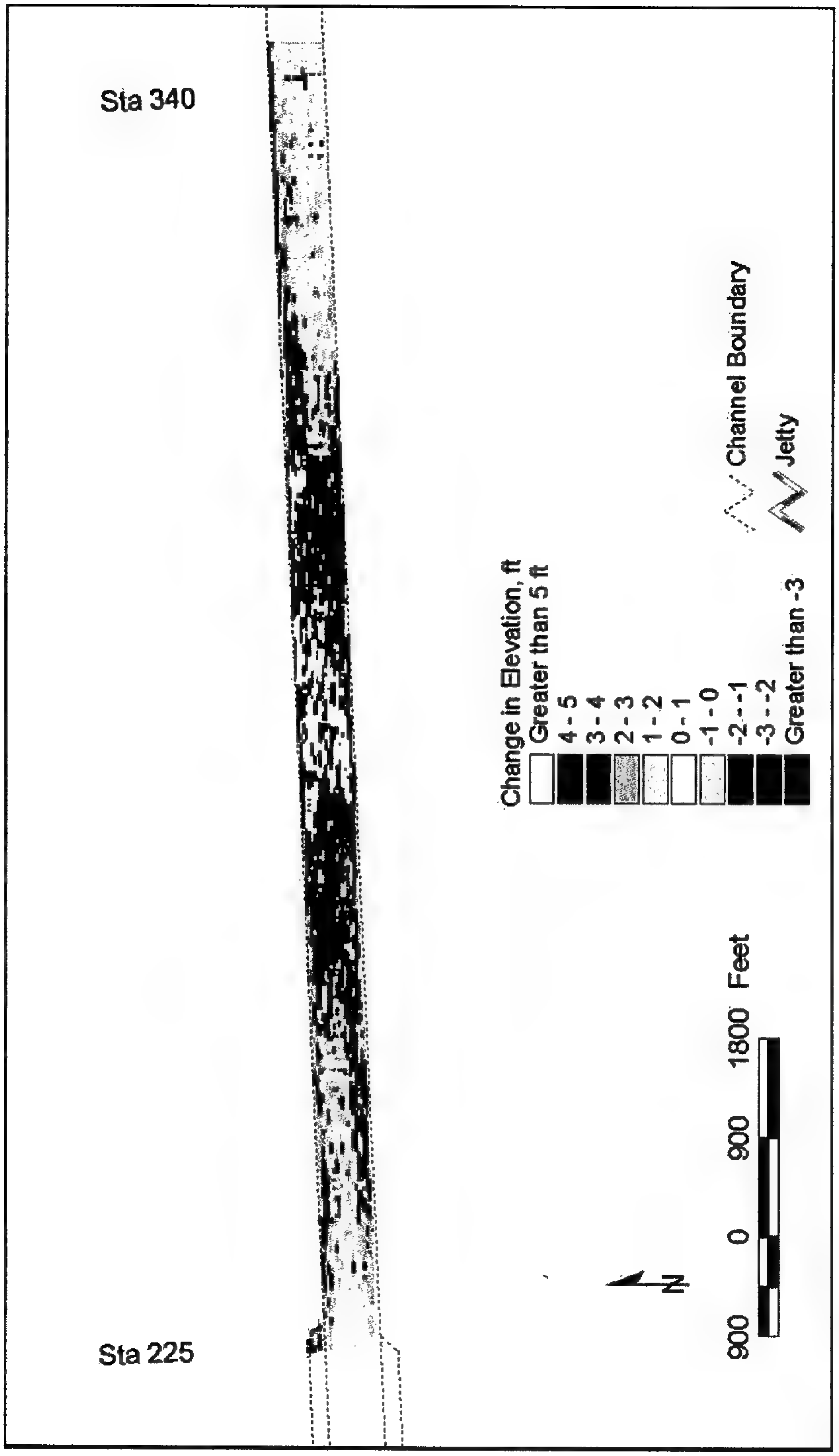

Figure 28. Elevation change between April 1996 and March 1997

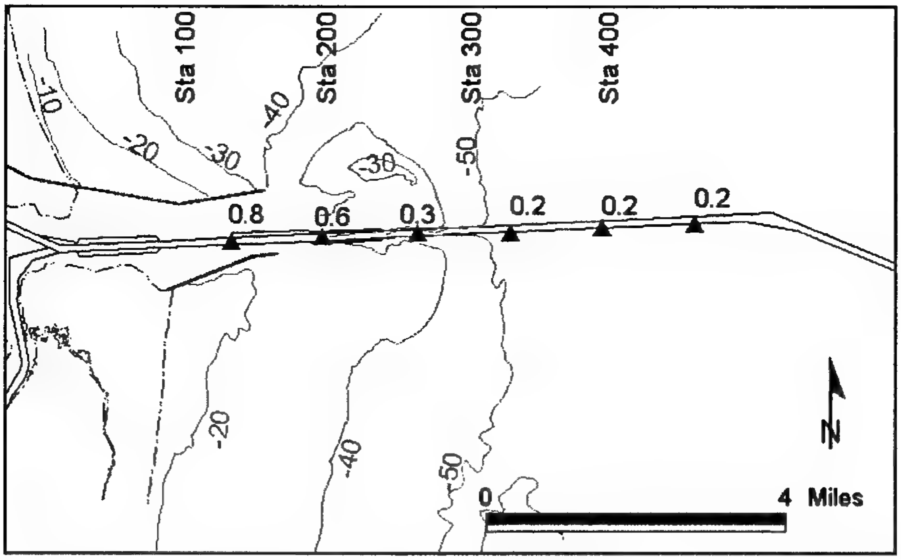

Figure 29. Ebb current velocity, m/sec

channel because the sediment originates from the east, and not from north or south of the inlet (Figures 30 and 31).

Seaward of sta 350, where depths exceed 50 ft mllw, shoaling may occur during storms with waves of large height and long period. Sta 350 is located 6.5 miles offshore, well seaward of the typical surf zone, so wave-induced longshore transport is rarely a factor. The shoaling rate between sta 345-375 was 1,200 cu yd/year/200 ft (Table 3).

Calculated shoaling rates

The shoaling rates described here were derived based on changes between bathymetric surveys provided by the Jacksonville District. The volume calculations were completed within the DMS volume calculator (Craig et al. 2001). This volume represents the amount of sediment between an arbitrary reference plane, taken as -100 ft mllw, and a triangulated irregular network (TIN) created from the bathymetric channel survey. The volume calculation was completed on a 5-ft grid. The bathymetric soundings were spaced approximately 20 ft apart across the channel and 100 ft apart along the channel.

A sensitivity analysis determined that the grid size should be 5 ft to minimize errors that might be introduced along the channel slope. The shoaling analysis covers from the throat of the inlet east 9.5 miles, because this area had the most survey coverage and experiences the most dredging. Comparison of shoaling rates is appropriate only within the same calculation because the area covered by each calculation may be different. A calculation that covers a larger area will have greater shoaling rates even if the change in surface elevation is the same. The calculations were not restricted to the minimum extent of all the surveys because this would eliminate the outer edge of the channel where most of the shoaling takes place. In a comparison of rates determined from different calculations, relative changes in shoaling rate should be examined. For example, which stations have the greatest shoaling rates in each calculation?

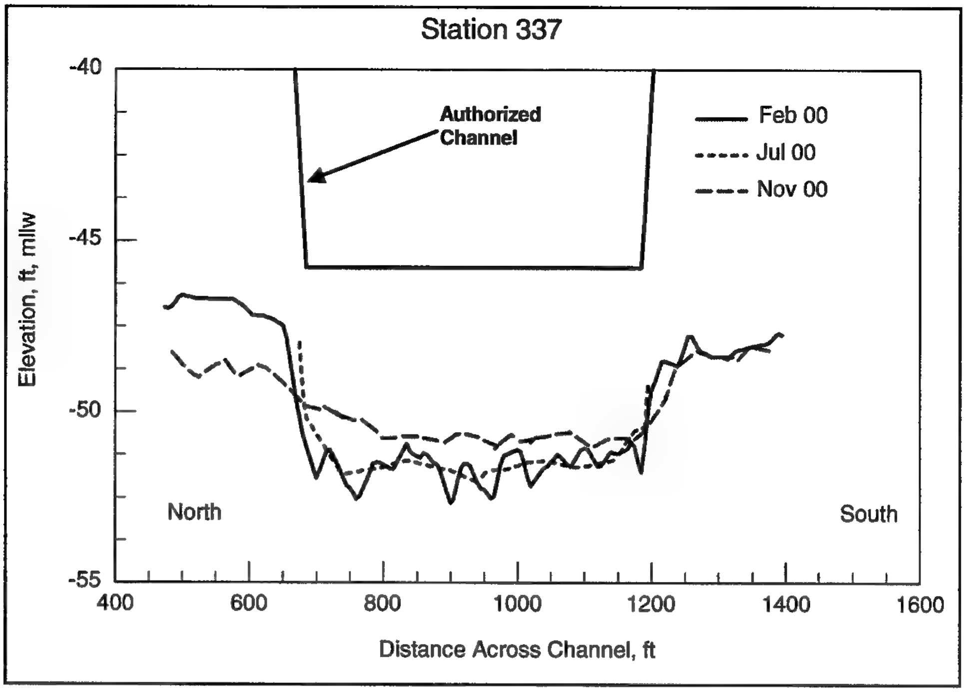

Figure 30. Channel cross section at sta 337

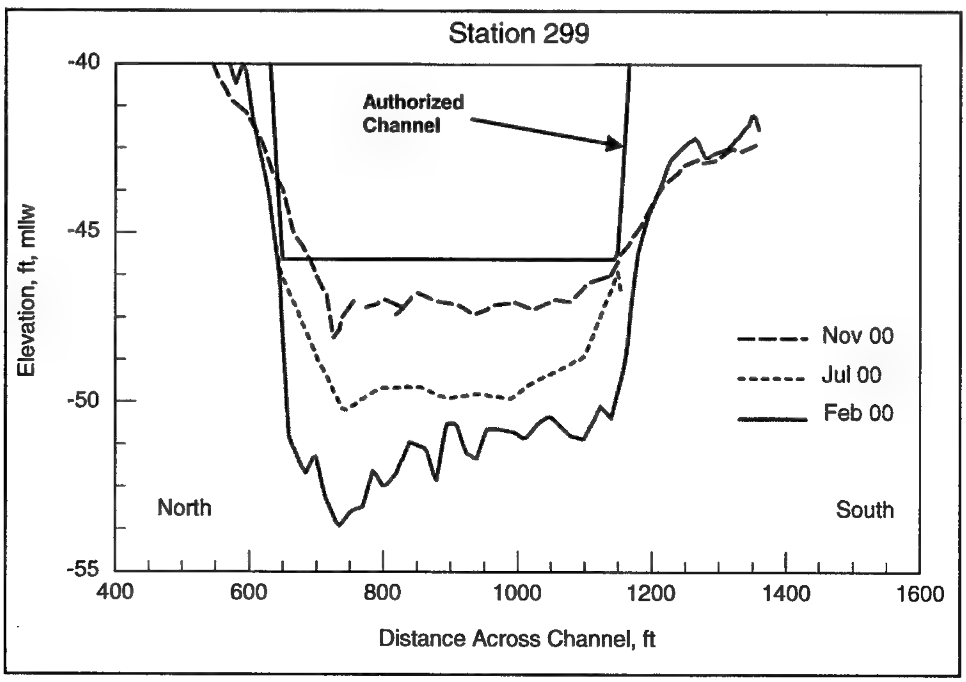

Figure 31. Channel cross section at sta 299

The Jacksonville District provided 16 bathymetric surveys between 1995 and 2000. Before the surveys were analyzed, the dates and spatial coverage of each survey were reviewed. Only surveys that were not temporally separated by a dredging event and had at least some common spatial coverage could be compared. There were seven dredging events between 1995 and 2000. These surveys could be divided into six sequences, each sequence separated by dredging. After each survey was evaluated, three shoaling rates could be found: 11 July 2000 to 3 November 2000, 15 March 1997 to 14 July 1997, and 8 April 1996 to 5 March 1997 (Figure 32).

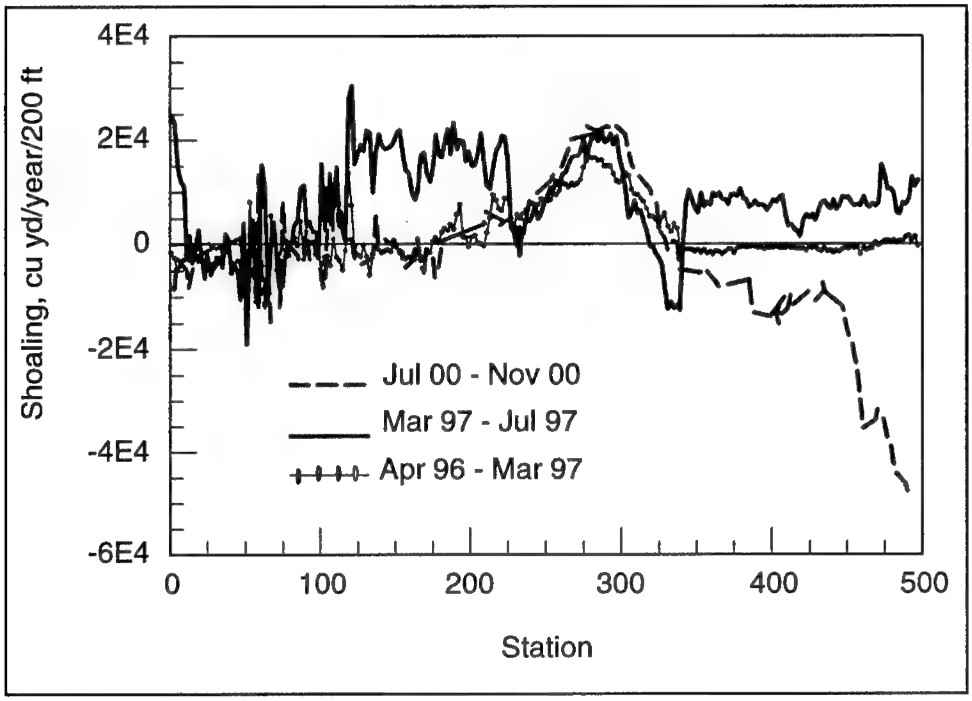

Figure 32. Shoaling rates by station

Channel Wideners

As part of the Trident channel expansion in 1987, settling basins, or channel wideners, were dredged to the north and the south of the authorized channel (Figure 33). There is a 300-ft wide extension north of the channel between sta 120 and sta 177 and a 150-ft-wide extension located north and south of the channel between sta 177 and sta 227. These basins were designed to trap sediment before it enters the channel and compromises navigable depth. The wideners are expected to increase the time between dredging.

This section examines the effectiveness of the channel wideners based on the amount of sediment deposited in the basins. The volume of sediment considered here is within the specified channel widener and above -51 ft mllw. This elevation was chosen because it is the elevation to which the channel is maintained; therefore, after the channel is dredged, the bottom elevation is at least at this level. The time associated with each volume is the time from the end of the

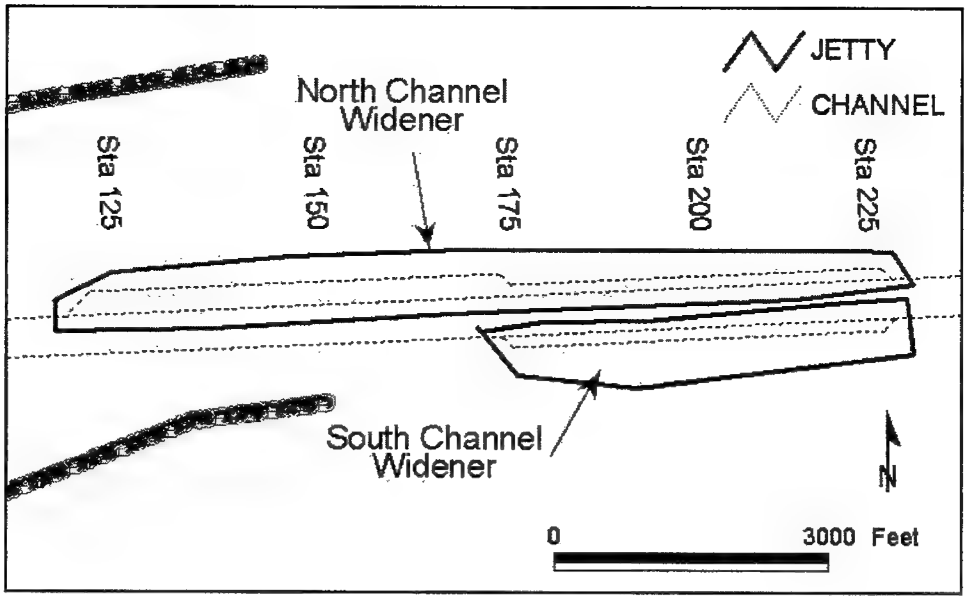

Figure 33. Location of channel wideners

last dredging event to the start of the survey. Post-dredging surveys were not analyzed because there is minimal time between the dredging and the survey. From 1996 to 2000, seven surveys covered the channel wideners and did not follow a dredging event.

Table 4 summarizes the results of the channel widener analysis. Similar calculations were done for the north and south wideners. First, the volume of sand in each widener and channel was found from the bathymetric survey and a base reference plane of –51 ft mllw. Then, the change in elevation (el) in the widener (Equation 1) and channel (Equation 2) was estimated by dividing the volume of sediment by the respective area:

$$\Delta\ (\text{el in widener}) = \frac{\text{vol. of sed. in widener}}{\text{Area of widener}} \tag{1}$$

$$\Delta\ (\text{el in channel}) = \frac{\text{vol. of sed. in channel}}{\text{Area of channel}} \tag{2}$$

where the symbol Δ denotes a change in the quantity in parentheses.

The condition of the channel without the widener was predicted by assuming that all the sand collected in the widener would have been deposited into the channel if the widener had not been there. The predicted change in channel elevation Δ(pce) is estimated in Equation 3:

$$\Delta(\text{pce}) = \frac{\text{vol. of sed. in widener}}{\Delta(\text{el in channel})} \tag{3}$$

Table 4
Channel Widener Analysis

Dredging Finished	Survey Started	Volume of Sediment above -51 ft mllw in Widener, cu yd	Volume of Sediment above -51 ft mllw in Channel, cu yd	Change in Widener Elevation, ft	Change in Channel Elevation, ft	Predicted Change in Channel Elevation due to Sediment in the Widener, ft	Time to Dredge with Channel Wideners, months	Time to Dredge Without North or South Wideners, months
North Widener								
Mar 96	Sep 96	434,641	940,048	1.6	1.5	0.7	12.3	7.4
Mar 96	Dec 96	408,942	885,020	1.5	1.4	0.7	18.4	10.9
Mar 96	Mar 97	397,862	733,551	1.5	1.2	0.7	31.1	17.5
Mar 97	Jul 97	562,678	1,174,819	2.1	1.9	0.9	6.2	3.7
Feb 99	Jul 99	961,000	1,406,443	3.5	2.3	1.6	6.5	3.3
Mar 00	Jul 00	979,007	166,757	3.6	0.3	1.6	44.0	4.9
Mar 00	Nov 00	1,003,935	200,493	3.7	0.3	1.6	71.6	9.3
	Average	678,295	786,733	2.5	1.3	1.1	27.1	8.1
South Widener								
Mar 96	Sep 96	188,719	940,048	2.3	1.5	0.3	12.3	7.4
Mar 96	Dec 96	191,783	885,020	2.3	1.4	0.3	18.4	10.9
Mar 96	Mar 97	169,131	733,551	2.0	1.2	0.3	31.1	17.5
Mar 97	Jul 97	243,582	1,174,819	2.9	1.9	0.4	6.2	3.7
Feb 99	Jul 99	417,928	1,406,443	5.0	2.3	0.7	6.5	3.3
Mar 00	Jul 00	365,580	166,757	4.4	0.3	0.6	44.0	4.9
Mar 00	Nov 00	341,701	200,493	4.1	0.3	0.6	71.6	9.3
	Average	274,061	786,733	3.3	1.3	0.4	27.1	8.1

NOTE: The number of significant digits in the third and fourth columns does not imply physical accuracy.

Two estimates for the time between dredging are presented, one with the widener (Equation 4) and one without (Equation 5). The calculated time between dredging was based on the time it should take the channel to shoal 3 ft, because 3 ft of advance dredging is presently allowed:

$$\Delta t(\text{with wideners}) = \frac{3\text{ft}}{\Delta(\text{el in channel})} * t \tag{4}$$

$$\Delta t(\text{without wideners}) = \frac{3\text{ft}}{\Delta(\text{el in channel}) + \Delta(\text{pce})_{\text{north}} + \Delta(\text{pce})_{\text{south}}} * t \tag{5}$$

where Δt equals the estimated time between dredging, and t equals the actual time between the beginning of the survey and the last dredging event, the first column of Table 4.

In the time estimate including the wideners, only the sediment found within the channel was supplied to the channel. In the time estimate without the wideners, the sand found in the wideners is also added to the channel. Without the wideners, channel elevation rises more quickly. The elevation changes and the predicted time between dredging events are based on the assumption of uniform change in depth across the channel. In reality, this may not be the case (e.g., see Figure 24), and areas with localized rapid shoaling may instead dictate dredging maintenance rather than the general trends discussed in the following paragraphs.

Both the north and south channel wideners shoal more, if normalized by area, than the channel; therefore it can be concluded that the wideners are functioning as intended. The north and south channel wideners shoaled an average of 2.5 and 3.3 ft, respectively, whereas the channel shoaled only 1.3 ft over the same time span. This disproportionate accumulation of sediment in the wideners indicates that the wideners are acting as designed and preventing some amount of sediment from reaching the channel.

The channel wideners are effective only because sediment is supplied from the north and the south along this section of channel. If the sediment were transported from the west by the ebb current, then the wideners would not be expected to decrease the maintenance requirements of the channel. The predicted time between dredging events decreases if the wideners are removed from the calculation, because the sediment contained within the wideners would now reach the channel, causing the shoaling rate to increase.

The estimated time between dredging shows a large variation, from 6.2 to 71.6 months with the channel wideners, and from 3.3 to 17.5 months without the wideners. Such variability is expected to be related to the frequency and magnitudes of storms for a given time interval, as well as antecedent morphology. For example, a particular storm may create a shoal close to the channel that subsequently deposits material into the channel during typical wave or storm conditions that occur much later. A dredging interval of less than 7.5 months poses a limitation to the Jacksonville District because dredging cannot be scheduled between 15 April and 1 December in accordance with environmental restrictions for sea turtles. Therefore, maintenance practice must allow the channel to afford navigable depth for at least 7.5 months.

Identification of Areas with High Shoaling Rates

Analysis of channel shoaling characteristics and dredging patterns was described in the previous sections. Here, the performance of the channel is evaluated based upon the volume of sediment dredged, as well as the shoaling rate. Channel performance was rated as good, fair, or poor. A rating of good indicates that minimal dredging is required and that the shoaling rates are low. A rating of poor describes areas that need the most maintenance dredging and have high shoaling rates. A rating of fair describes transitional areas between good and poor.

The channel was divided into four sections based on the rating criteria (Table 5, Figures 34 and 35). West of sta 100 and east of sta 340, the channel has had good performance because there is little shoaling, and maintenance requirements are minimal. The section from sta 0 to 100 has a large sediment supply and strong tidal current. The strong current removes sediment from the channel and prevents shoaling. The stretch from sta 340 to 500 has weaker currents, and the sediment supply is also lower, so there is minimal shoaling in this area. Between sta 100 and 225, channel performance declines. This section has an overall rating of fair.

The dredging maintenance and shoaling rate both increase to the east as the ebb tidal current weakens. Weaker currents are less capable of scouring the channel or transporting sediment from it. Sta 225 through 340 have high shoaling rates and, therefore, require the most dredging. This section of the entrance has shown poor performance. Deposition of fine-grained sediment seaward of the ebb shoal is the main cause of shoaling in this area.

Table 5
Description of Channel Maintenance[1]

Stations	Maintenance Performance Rating	Shoaling Rate cu yd/year/200 ft	Dredged Volume cu yd/100 ft
0-100	Good	-2,200	1,100
100-225	Fair	5,700	26,700
225-340	Poor	12,000	67,000
340-500	Good	-3,800	5,600

[1]The shoaling rates and dredged volume for each section of channel were calculated as the average of either parameter for each station within that section excluding the first ten and last ten stations. The first and last ten stations of each section were omitted from the averages because they were considered transitional and did not necessarily represent the characteristics of the subject section. All values were rounded to the nearest hundred. A minus value for the shoaling rate signifies an overall loss of material from that section.

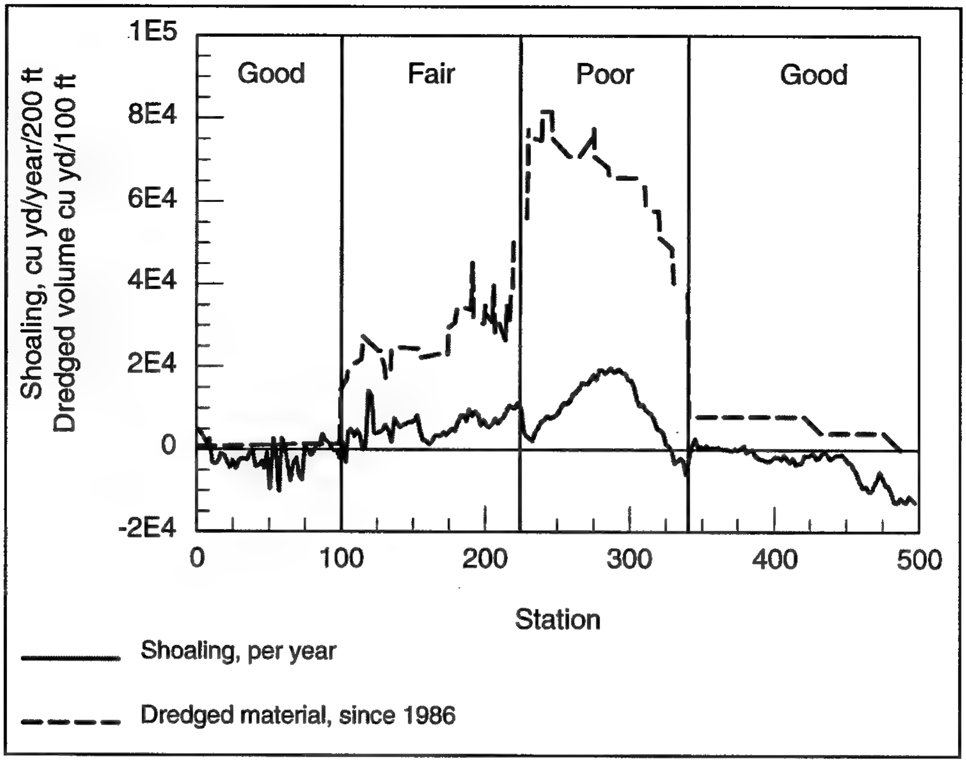

Figure 34. Channel performance rating

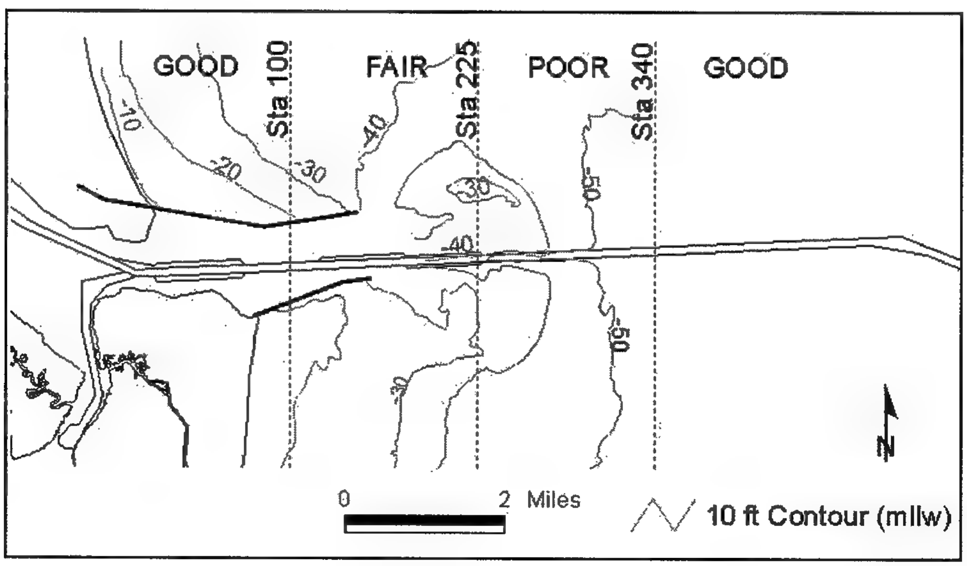

Figure 35. Plan view map of channel performance rating

3 Wave Analysis

This chapter describes the transformation of deep-water waves as they approach the southern end of Cumberland Island and the northern end of Amelia Island. Potential longshore sand transport rates are developed from this information to supplement the geomorphic analysis given in the previous chapter.

Wave measurements and hindcasts for the Kings Bay area are first described. These data serve as the boundary conditions to initialize the STeady-state spectral WAVE transformation model (STWAVE) (Smith, Resio, and Zundel 1999). Next, development of the bathymetric grid is documented. Finally, the calculated potential longshore sand transport rates are presented, with their distribution across the surf zone given as a climatological estimate of longshore transport for the surf zone adjacent to the north jetty.

Wave Data

To estimate wave-induced longshore sand transport from wave information, it is necessary to drive a wave transformation model with directional wave data (wave height, period, and direction). Three wave sources were accessed in this project: National Data Buoy Center (NDBC) buoy data, Coastal Data Information Program (CDIP) pressure gauge array data, and CHL's WIS Atlantic Ocean hindcast information. The CDIP data were furnished by the CDIP operated by the Scripps Institution of Oceanography under sponsorship of the USACE and the California Department of Boating and Waterways. Hindcast wave information was acquired from a new version of the WIS now under production. The locations of the gauges are shown in Figure 36.

The NDBC deployed and operated directional wave buoy 41008 offshore of the project area from 1988 to 1992. During this time, the heave-pitch-roll buoy was located about 17 miles east of St. Marys Entrance at about 30.7°N, 81.1°W. Wave height data were available from the NDBC, but not the directional information. The wave directional analysis for this buoy was completed at CHL based on information provided by NDBC. In 1997, the buoy was redeployed at 31.4°N, 80.9°W. These data are not included in the wave modeling because there was not enough information to determine wave direction reliably. Table 6 summarizes the data available from NDBC Buoy 41008.

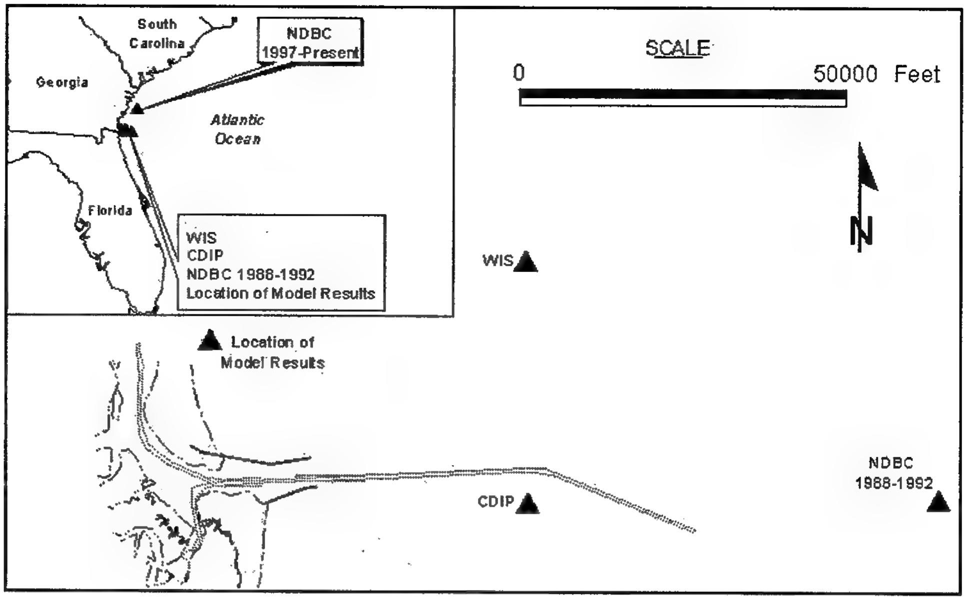

Figure 36. Locations of wave gauges and wave model output

Table 6
Data Availability from NDBC Buoy 41008

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1992	x	x	x	x								
1991	x	x	x	x	x	x	x	x	x	x	x	x
1990	x	x	x	x	x	x	x	x	x	x	x	x
1989	x	x	x	x	x	x	x	x	x	x	x	x
1988			x	x	x	x	x	x	x	x	x	x

Measurements are available since 1995 from the CDIP pressure gauge array (sta 08301) located approximately 5 miles east of St. Marys Entrance. The pressure gauges are mounted on the corners of a triangular platform, shown in Figure 37. The platform is located at 30.7°N, 81.3°W in a water depth of about 15 m mhw. The instrument reports wave energy, peak wave period, mean wave direction, and atmospheric pressure. The data include wind and wave parameters, but not complete directional spectra. Table 7 summarizes the data available from CDIP sta 08301 that are applicable to this project.

The WIS has made available a new hindcast for the U.S. Atlantic Coast. The hindcast applied CHL's second-generation, time-dependent directional spectral wave model driven by newly developed, high-quality wind fields. Information from WIS output point number 143, located at 30.8°N, 81.3°W in a water depth of 12 m mhw was selected for this project (Figure 36). The WIS hindcast included the years 1995, 1996, 1997, and 1999 and contains complete directional wave spectra at 22.5-deg resolution, as well as wind speed and direction.

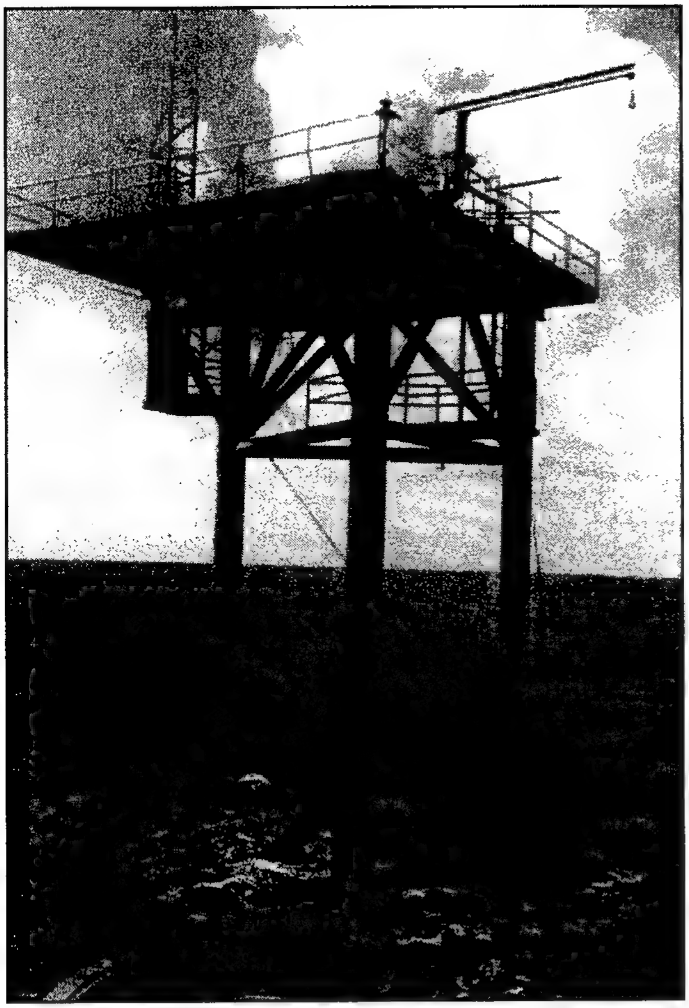

Figure 37. CDIP sta 08301

Table 7
Data Availability from CDIP Station 08301

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
2001	x	x	x	x	x	x	x	x	x			
2000	x	x	x	x	x	x	x	x	x	x	x	x
1999	x	x	x	x	x	x	x	x	x	x	x	x
1998	x	x	x	x	x	x	x	x	x	x	x	x
1997	x	x	x	x	x	x	x	x		x	x	x
1996	x	x	x	x	x	x	x	x		x		x
1995								x			x	x

Bathymetry Data

Transformation of deep-water waves into shallow water changes the wave height and direction according to depth. Therefore, accurate representation of the nearshore bathymetry is essential for determining the characteristics of nearshore waves, in particular, near inlets (Grosskopf et al., in preparation). Local bathymetry for the nearshore area of Cumberland Island, St. Marys Entrance, and Amelia Island was developed from a variety of sources. Adjustments to the underlying data sources were carefully applied to achieve a consistent vertical reference datum throughout the bathymetric grid. Additionally, the extent and density of the underlying data had to be maximized to assure that relevant shoals were resolved. Many shoals exist on the continental shelf seaward of St. Marys Entrance and along Cumberland Island and Amelia Island that could modify the wave transformation. The bathymetric grid is bounded on the landward margin by the high-water line (+0.3 m mhw) and contains approximately 117,000 data points. Figure 38 shows a contour plot of the final grid for the vicinity of St. Marys Entrance.

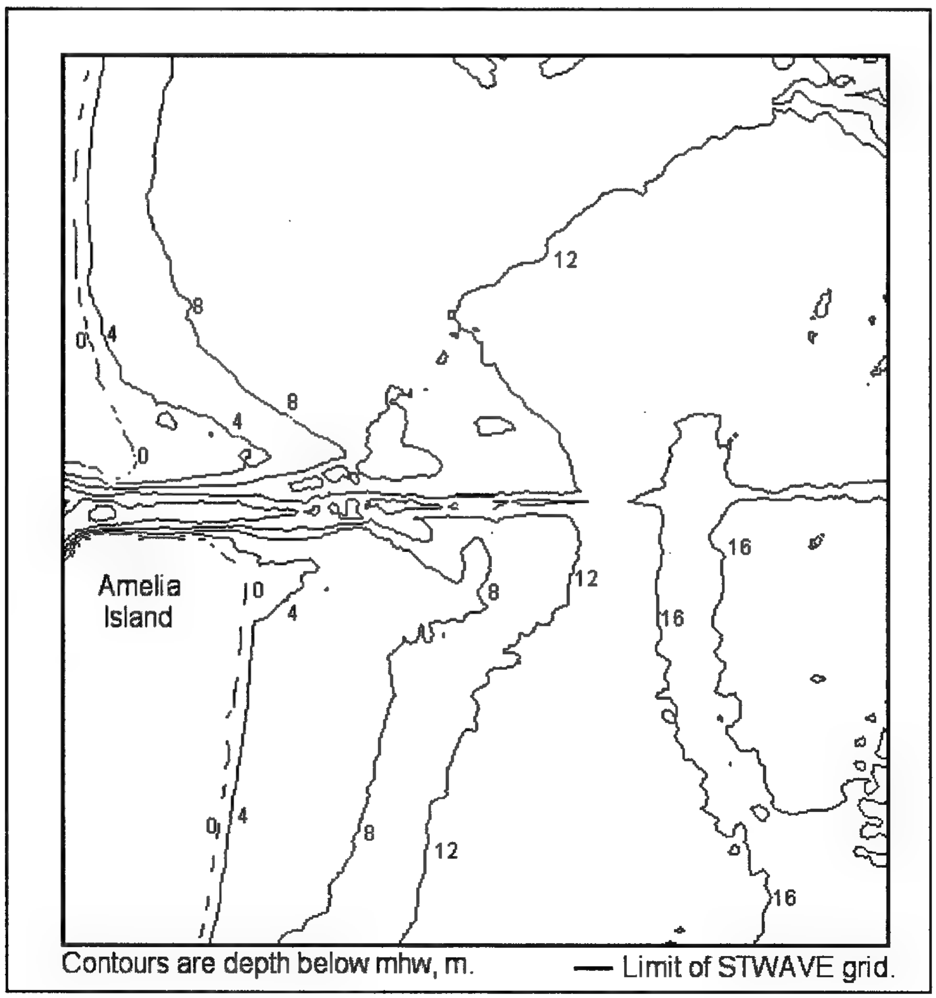

Figure 38. Nearshore bathymetry, vicinity of St. Marys Entrance

The data sets composing the bathymetry were collected between 1954 and 2000. Most of the data was collected between 1974 and 1979. All data were converted from the original vertical datum to mhw. Relevant data sources are as follows:

a. November 2000 navigation channel survey from the Jacksonville District.

b. Ebb-tidal shoal surveys from 1979 and 1992.

c. Bathymetric surveys offshore of Cumberland and Amelia Islands from 1974 and 1977.

d. Local surveys updrift and downdrift of the entrance channel jetties from 1954 and 1955.

e. NOAA Navigation Chart 11503 for areas not covered by the local surveys. The data from this source were taken from the grid composing the tidal circulation model described in the next chapter.

The local model domain and bathymetry are illustrated in Figure 38. The grid is rotated such that waves enter from easterly quadrants, with waves from the east traveling normal to the offshore boundary of the grid. Grid dimensions are 150 cells by 160 cells, with a cell size of 100 m.

Wave Model

A TMA spectrum was generated for the NDBC and CDIP wave parameter input records (combination of height, period, and direction) with a cosine-to-the-fourth directional energy spreading and 5-deg directional spectral resolution. WIS hindcast input wave conditions were taken directly from the onshore direction bins of the energy spectrum produced by the hindcast. Wind was applied to the modeled domain as reported by the measurements, so the simulations are representative of wave generation, propagation, and transformation from the ocean boundary of the grid toward the coast and inlet. For simulated times for which wind information was not available, a 10-knot wind was applied to the water surface in the direction of the peak wave energy.

The 100-m STWAVE grid was developed to resolve the nearshore bathymetry and other features adequately. Directional spectra were synthesized with a 5-deg directional resolution to better represent the nearshore wave transformation. These conditions caused run times to be relatively prohibitive. Several attempts to reduce calculation time were made, while still meeting study objectives. First, the grid resolution was reduced to 300 m, but the nearshore refraction was underestimated, so the original 100-m grid was restored.

Because the calculation could not be simplified by reducing the grid resolution, the focus turned to summarization of the wave data. To retain an accurate representation of wave transformation and directional resolution, a daily average was calculated to develop a representative wave condition from the CDIP 3-hr parameter records. If data were missing or bad, those points were estimated by a linear vector (including direction) average of the surrounding data. A daily peak wave period was also generated by means of this method. The synthetic spectrum could then be discretized at a 5-deg and 0.01-Hz resolution

and transformed at the 100-m grid resolution. The CDIP data were filtered prior to processing to eliminate outlying data and waves heading in highly oblique or offshore directions that would contaminate the daily representative shore-directed wave calculation. In the case of highly oblique or offshore-directed waves, the waves were zeroed because they vary erratically and, in principal, do not contribute to the longshore transport. These waves are typically low in height.

A detailed examination of one month, June 1996, was completed to compare the differences in the various wave conditions resulting from the different inputs to STWAVE. Three wave inputs were compared – the raw data, the daily average, and the daily average of the energy-flux weighted wave data, following concepts given in Kraus and Harikai (1983). The three data types were compared only for the CDIP gauge. The daily average is the average of the eight 3-hr data points for each day after the raw data had been filtered of all waves outside of ± 45 deg from the grid-normal direction. The third wave spectrum was developed by weighting the data by the longshore energy flux factor P_{ls} given by

$$P_{ls} = (EC_g)_b \sin 2\alpha \tag{6}$$

where

E = total energy in one wavelength per unit crest width

C_g = wave group velocity

α = angle the wave crest makes with the trend of the shoreline (Shore Protection Manual (SPM) 1984).

Wave energy is defined as

$$E = \frac{1}{8}\rho g H^2 \tag{7}$$

where

ρ = the density of water

g = the acceleration of gravity

H = the wave height

The quantity P_{ls} was evaluated at the gauge to understand the sediment transport potential of the wave input. The P_{ls} weighting emphasizes those waves that are effective in contributing to longshore sediment transport. A P_{ls} value for each record was calculated, and the effective longshore energy contribution of each record was multiplied by the ratio of its P_{ls} to the total P_{ls} to define weighted wave height and period values:

$$H_{ls\,i} = H_i\left(\frac{P_{ls\,i}}{\sum P_{ls}}\right) \tag{8}$$

$$T_{ls\,i} = T_i\left(\frac{P_{ls\,i}}{\sum P_{ls}}\right) \tag{9}$$

where the subscript *i* denotes an individual record, and *T* equals the wave period.

Figures 39-41 illustrate the difference in quantities between the CDIP 3-hr data, the daily average, and the P_{ls}-weighted data. Figures 42-44 illustrate calculation results from the boundary conditions presented in Figures 39-41 at the location shown in Figure 36, in 6 m of water. Direct implementation of the 3-hr values produces a noisy record that would give unrealistically variable and large longshore transport rates.

The daily averages correspond well with the results of the P_{ls}-weighted method, which has direct bearing on the longshore sediment transport rate and, therefore, are considered reliable. In the following section, the longshore sand transport rate was calculated based upon the daily averages.

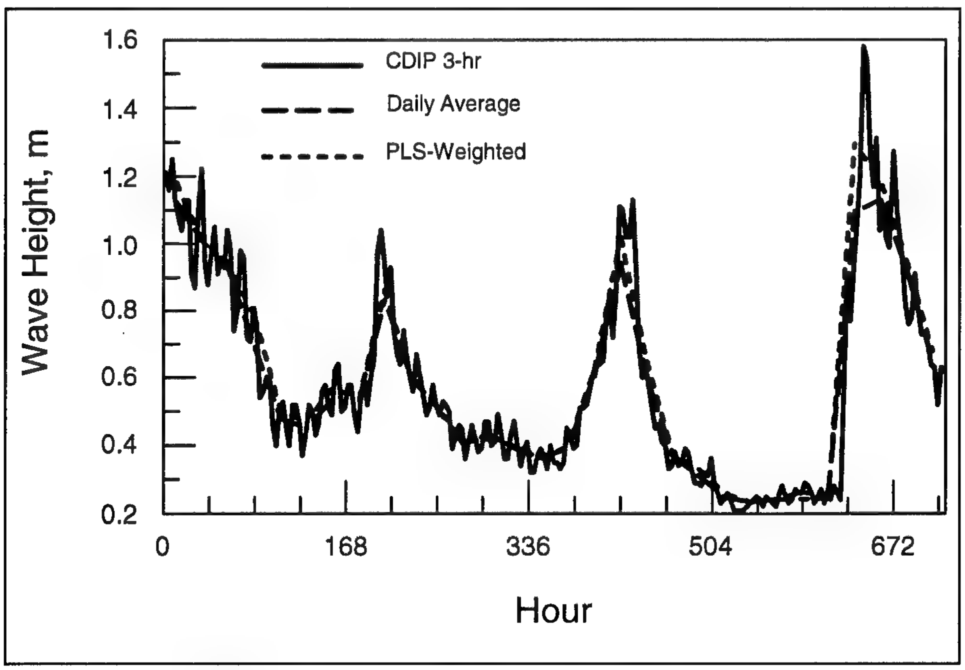

Figure 39. Comparison of wave height input, June 1996

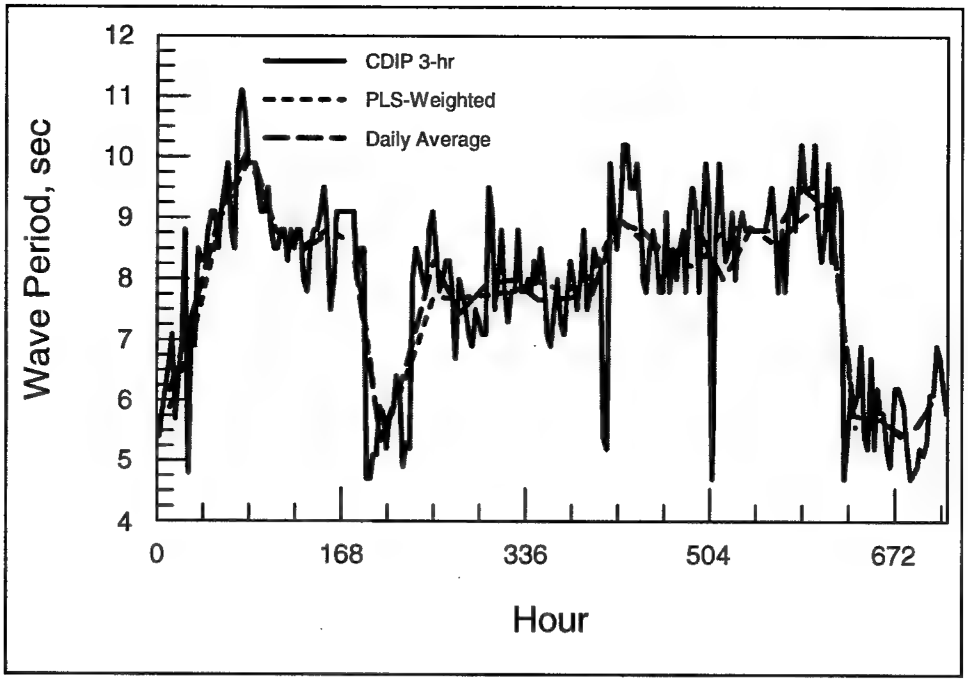

Figure 40. Comparison of wave period input, June 1996

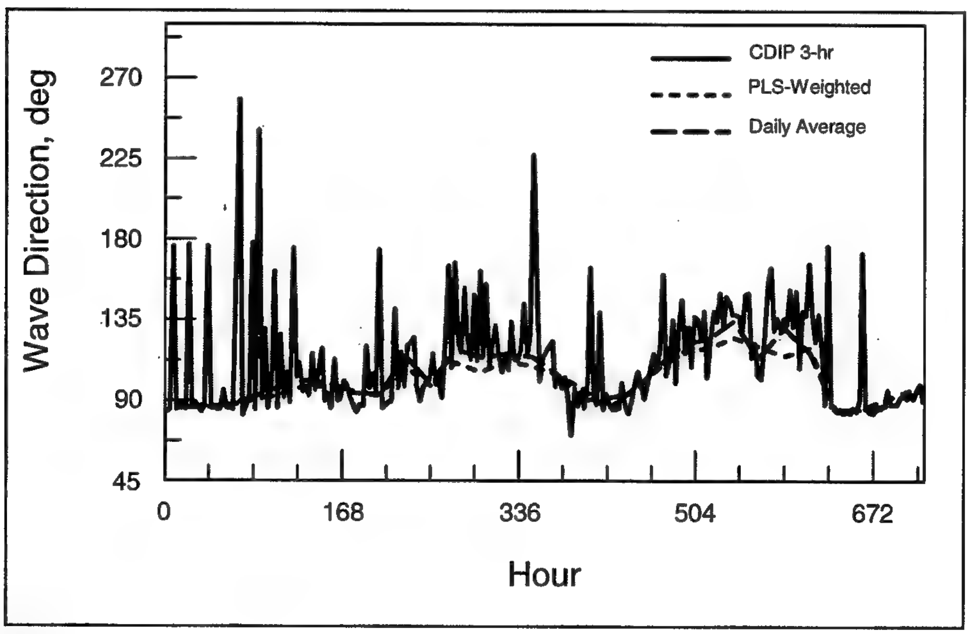

Figure 41. Comparison of wave direction input, June 1996

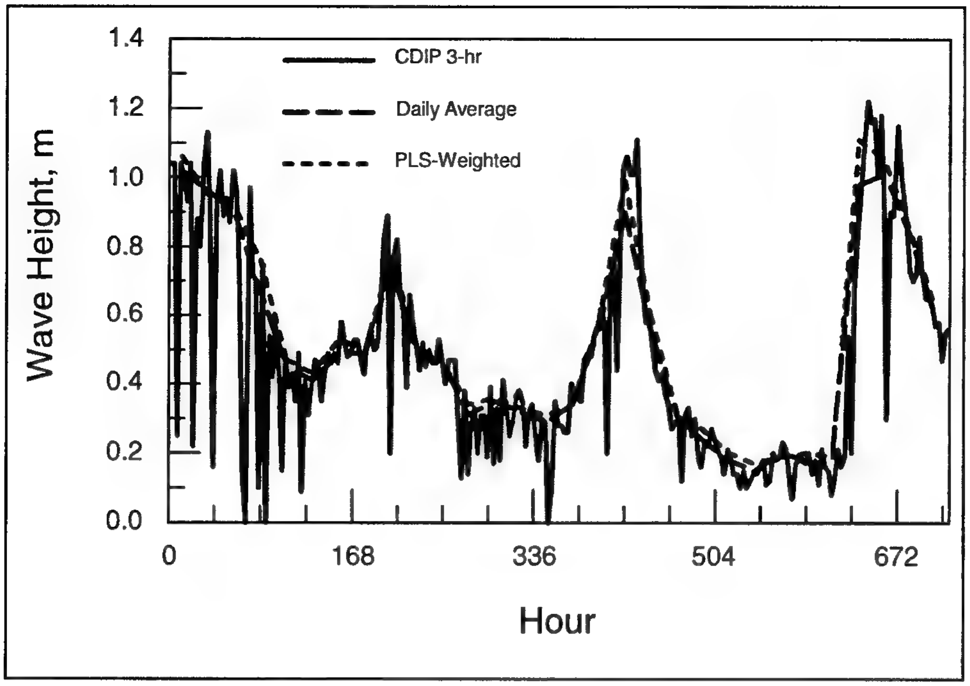

Figure 42. Comparison of modeled wave heights at the location shown in Figure 36 with different boundary conditions, June 1996

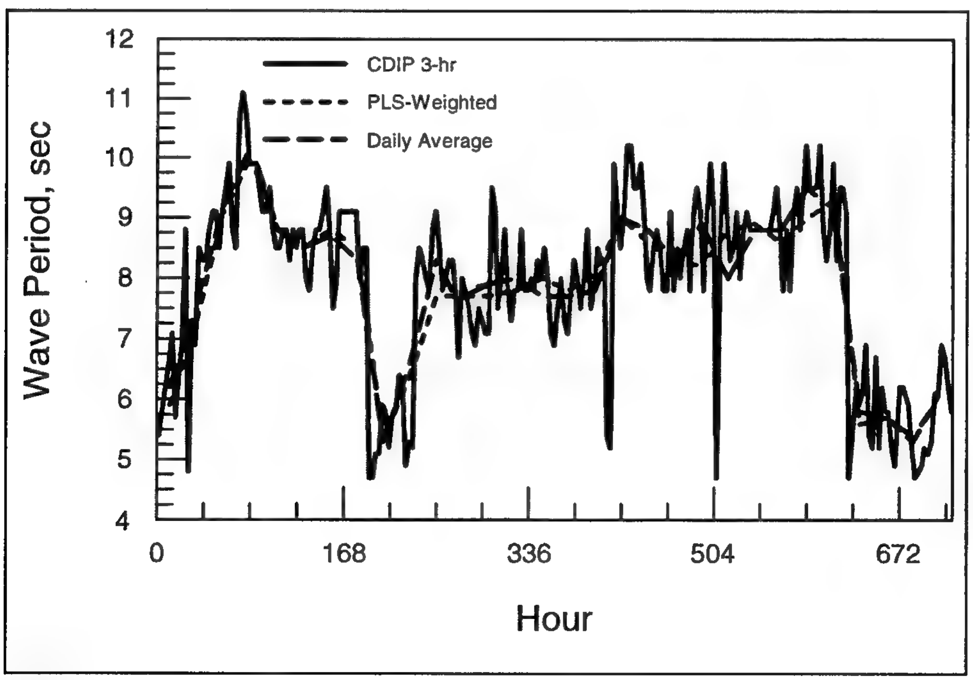

Figure 43. Comparison of modeled peak wave period at the location shown in Figure 36 with different boundary conditions, June 1996

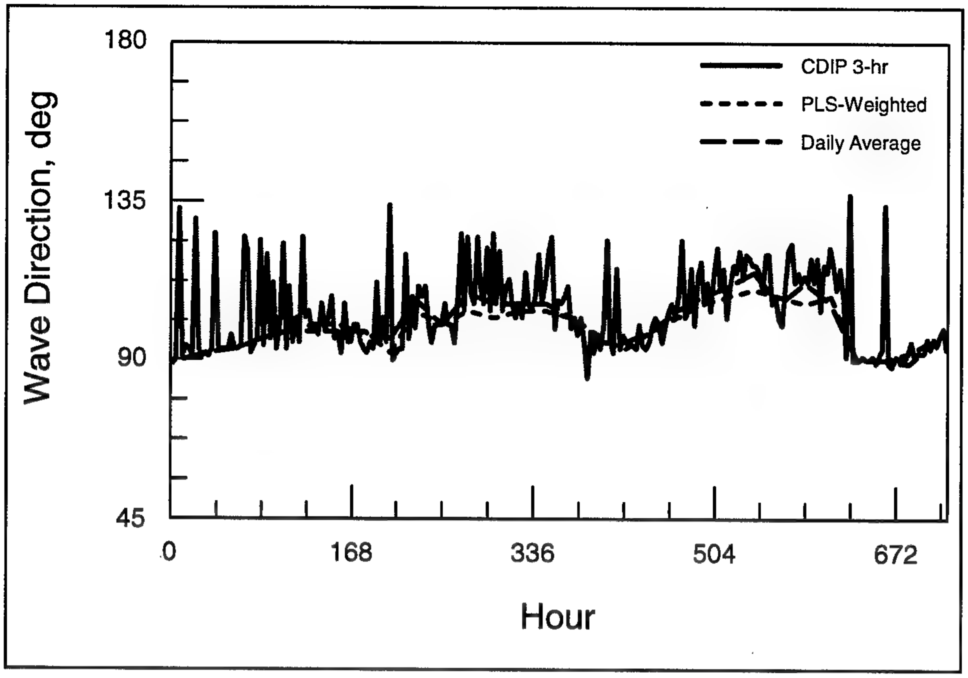

Figure 44. Comparison of modeled wave direction at the location shown in Figure 36 with different boundary conditions, June 1996

Longshore Sediment Transport

The amount of sediment transport alongshore depends on the height of the breaking waves and the angle of the approaching wave to the shoreline. Here, the beach sediment consists predominantly of sand. Longshore sand transport is confined mainly to the surf zone. Both the nearshore wave height and direction were calculated with STWAVE as described in the previous section. In this section, those results allow estimation of the potential longshore sand transport rate within the study area. The potential longshore sand transport rate Q was calculated with the following formula (SPM 1984, U.S. Army Engineer Waterways Experiment Station 1989):

$$Q = \frac{K}{(\rho_s - \rho) g a} P_{ls} \tag{10}$$

where

K = a nondimensional empirical sand transport coefficient

ρ_s = the density of the sand

a = porosity of the sand

Here, K was set to 0.39 for significant waves. The transport rate is termed the potential rate because adequate sand must be available to achieve this empirically determined quantity.

The potential transport rate was calculated at selected locations along the islands. The time-histories were post-processed by time-integrating the potential longshore sediment transport under the assumption of a fixed shoreline orientation.

The potential longshore sand transport rate for June 1996 was calculated to be 10,700 cu yd (Figure 45). Days 1-5 and days 27-29 have the strongest southern transport for that month (Figure 46). These days show peaks in the wave record, exceeding a height of 1 m, and waves approaching almost from due east. The easterly waves combine with the slightly northeast-southwest shoreline orientation to direct the longshore current to the south. The wave heights also increase on days 10 and 18, but these higher waves are associated with a more southerly direction. Therefore, the transport did not increase as much as in the beginning and end of the month.

Annual longshore transport rates

Wave time-histories were developed at the boundary of the STWAVE grid from the NDBC, CDIP, and WIS data sources. These input time-histories pertained to locations in 15 to 20 m of water. The STWAVE model then transformed those boundary wave conditions into the shallow water of the nearshore region, further accounting for shallow-water generation, propagation, and attenuation mechanisms. The result is a transformed time-history of wave height, period, and direction close to shore, nominally in about 6 m of water.

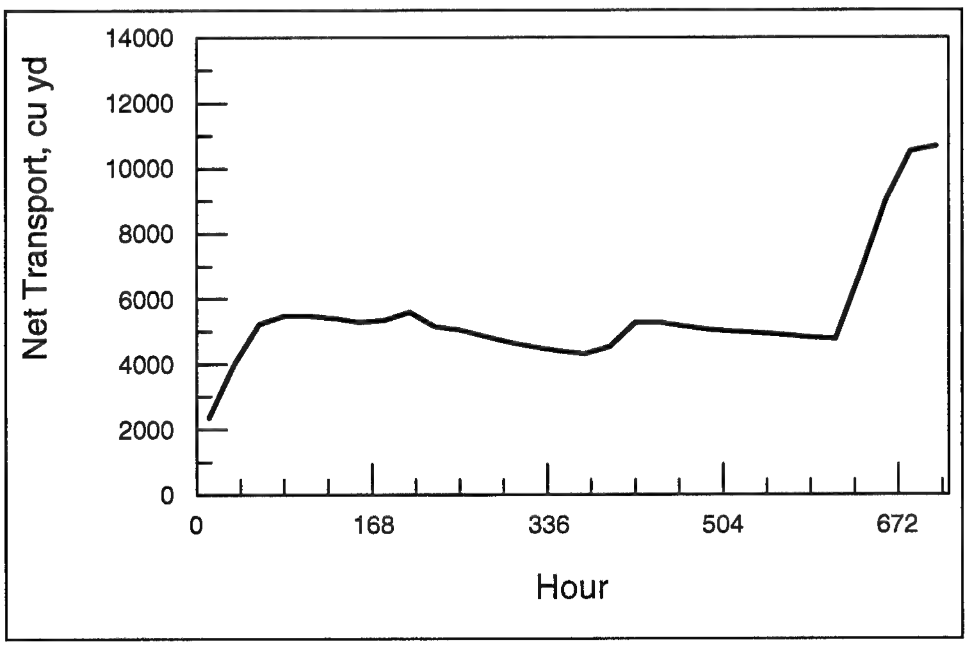

Figure 45. Cumulative net longshore transport potential (positive to the south) for June 1996 at the location shown in Figure 36

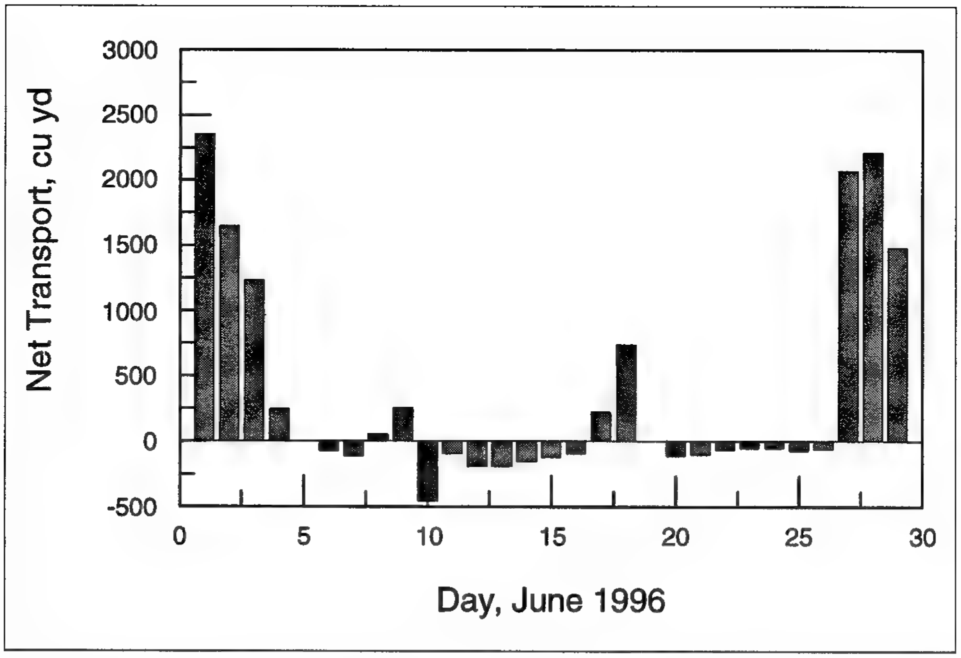

Figure 46. Daily net longshore transport potential rate at the location shown in Figure 36

Longshore transport rates, longshore variation

Potential longshore transport rates were calculated for the southern end of Cumberland Island and the northern end of Amelia Island, herein called the "regional" rates, with respect to a shoreline orientation that is representative of about 5 miles of shoreline in those areas. Rates were also calculated for three locations along Cumberland Island and two locations along Amelia Island, as shown in Figure 47. The Cumberland Island locations are 4.6, 3.3, and 2.1 miles north of the inlet. The Cumberland Island shoreline orientation varies considerably because of the pronounced fillet on the north side of the inlet. The Amelia Island locations are 3.2 and 4.5 miles south of the inlet. Northern Amelia Island exhibits relatively less change in shoreline orientation than Cumberland Island.

The shoreline orientation was varied slightly to assess the sensitivity of the longshore transport potential to uncertainty and to local changes in orientation. The results are summarized in Table 8. The table presents the net longshore transport potential rates for each year and for each data source. Characteristics of the calculated transport rates include the following:

a. A consistent trend of southerly transport exists, except at the north fillet where inlet bathymetry is likely dominant and where the shoreline orientation may not be accurately estimated.

b. Greater variability and sensitivity to shoreline orientation arise from estimating longshore transport rates from WIS information because of the coarse resolution of the input wave spectra.

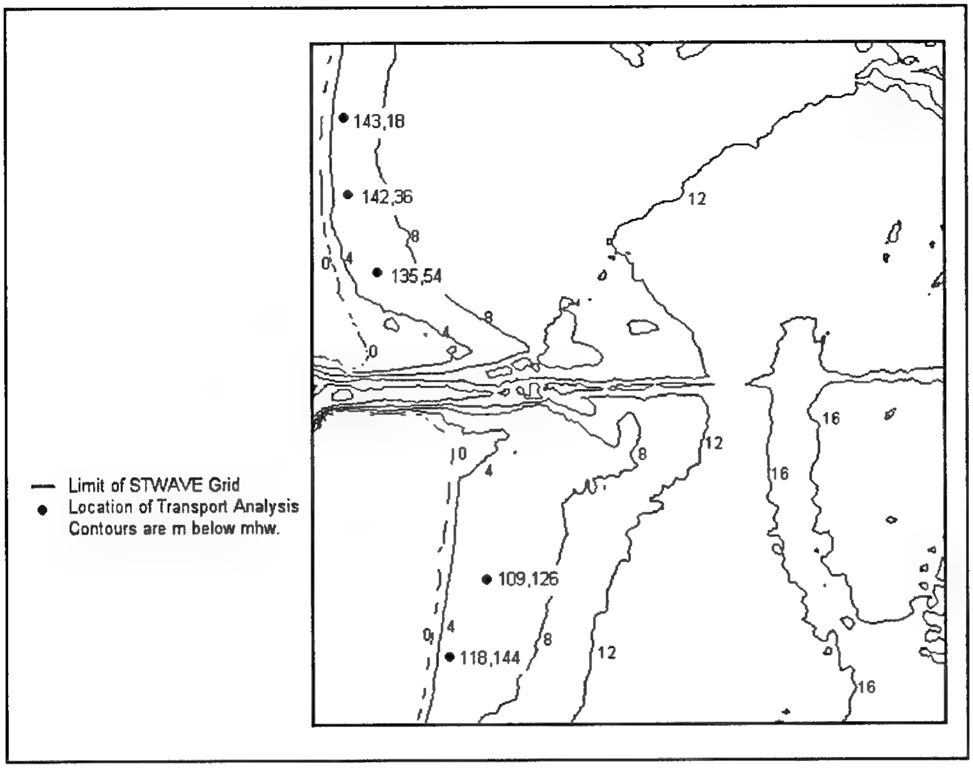

Figure 47. Location of analysis nodes

c. Greater consistency (less variability) is associated with estimates of longshore transport rates from CDIP data because the gauge is closer to shore and the synthetic wave spectra more smoothly represent the position and movement of the peak wave direction.

d. A nearly balanced, but slightly southerly, regional net transport rate is produced through boundary input of hindcast wave information, whereas a more southerly directed pattern is produced through input of the CDIP wave measurements.

e. The CDIP wave data produce more sensible net longshore sand transport rates, with a clear southerly directed rate of 98,000 to 288,000 cu yd/year throughout the region and nearly balanced transport in the north fillet. The estimate of 247,000 cu yd/year appears to be the most reasonable regional average (compare Table 3 for estimates found in the literature).

Longshore transport rates, cross-shore distribution

The longshore transport rate was analyzed as a function of distance from the shoreline at the southern end of Cumberland Island, Georgia. This area is expected to supply a substantial amount of littoral material to the navigation channel. The CDIP daily average wave data served as input, with a shoreline orientation of 10 deg, to estimate southerly directed transport that would enter the navigation channel at St. Marys Entrance. The waves were not transformed to

Table 8
Annual Net Transport Rates, cu yd, Based on STWAVE Transformation to Approximately 6-m Depth Contour (mhw), 100-m grid

Year	Regional Estimate	4 miles North of Inlet	2.9 miles North of Inlet	North Fillet	South Fillet	3.9 miles South of Inlet
Annual Net Transport Rates Based Upon WIS Input						
1995	-294,500	-209,500	-327,000	-961,500	189,500	-811,000
1996	359,500	163,500	59,000	-575,500	654,000	39,000
1997	196,000	150,500	46,000	-412,000	549,500	111,000
1999	78,500	-26,000	-111,000	-700,000	510,000	-484,000
Average	85,000	19,500	-83,500	-662,000	476,000	-286,000
Annual Net Transport Rates Based Upon CDIP Input						
1996[1]	202,500	104,500	131,000	-13,000	196,000	72,000
1997[2]	209,500	131,000	85,000	6,500	202,500	117,500
1998	189,500	144,000	72,000	-46,000	274,500	65,500
1999[3]	274,500	150,500	65,500	-72,000	386,000	78,500
2000	359,500	235,500	137,500	0	379,500	189,500
Average	247,000	153,000	98,000	-25,000	287,500	104,500
Annual Net Transport Rates Based Upon NDBC Input						
1989[4]	1,098,500	804,500	608,000	287,500	1,033,500	641,000
1990	673,500	510,000	314,000	26,000	817,500	301,000
1991	1,125,000	843,500	601,500	209,500	1,144,500	634,500
Average	965,500	719,500	508,000	174,500	998,500	525,500

NOTE: Negative rate denotes transport north.
[1] 1996 includes only 16 January - 27 July and 1-31 December
[2] 1997 includes only 1 January - 25 March and 1 October - 31 December
[3] 1999 has 6 days missing in January
[4] 1989 missing 12 days in October and 18 days in November

shallow water with STWAVE. Based upon a given wave height, period, and direction, and an assumed Rayleigh distribution of wave height, a breaking depth and associated location were calculated for each 0.5-m increment of the wave height distribution.

The locations of the actual transport calculations are shown in Figure 48. Figure 49 shows three representative profiles taken from the bathymetry file developed for this project. The cross-section locations are just north of the jetty. The first profile is located 1,000 ft north of the jetty, the second 980 ft north of the first, and the third 980 ft north of the second. The second cross section was selected for analysis. The high spot about 14,000 ft from shore is assumed to be a spurious data point. Based upon the measured wave direction and the energy flux method with a regional shoreline angle of 10 deg, the associated longshore transport was determined for each increment of wave height in a Rayleigh distribution. The process was repeated for each wave record in an annual time-history, and the southerly transport quantities were summed at each location along the beach profile.

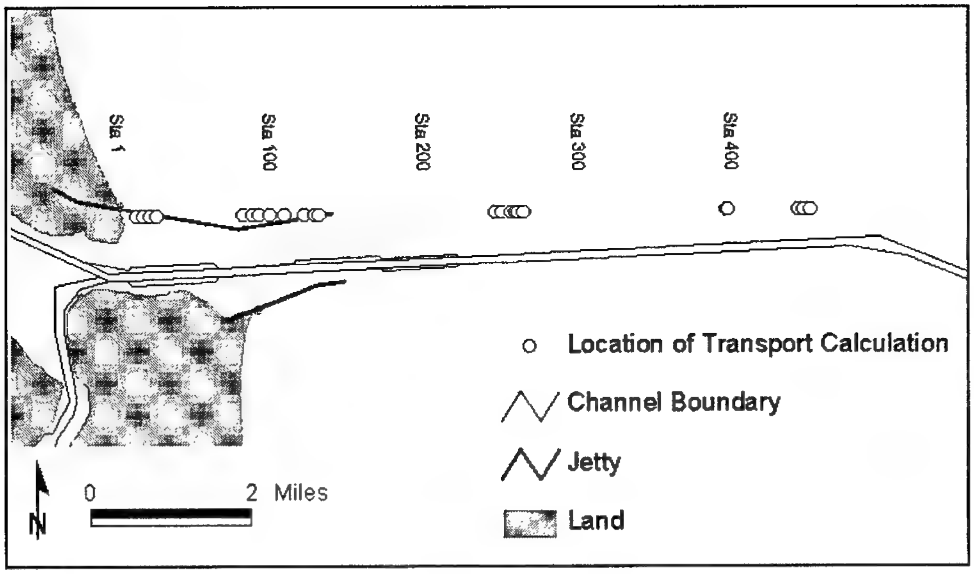

Figure 48. Location of southerly longshore transport calculation

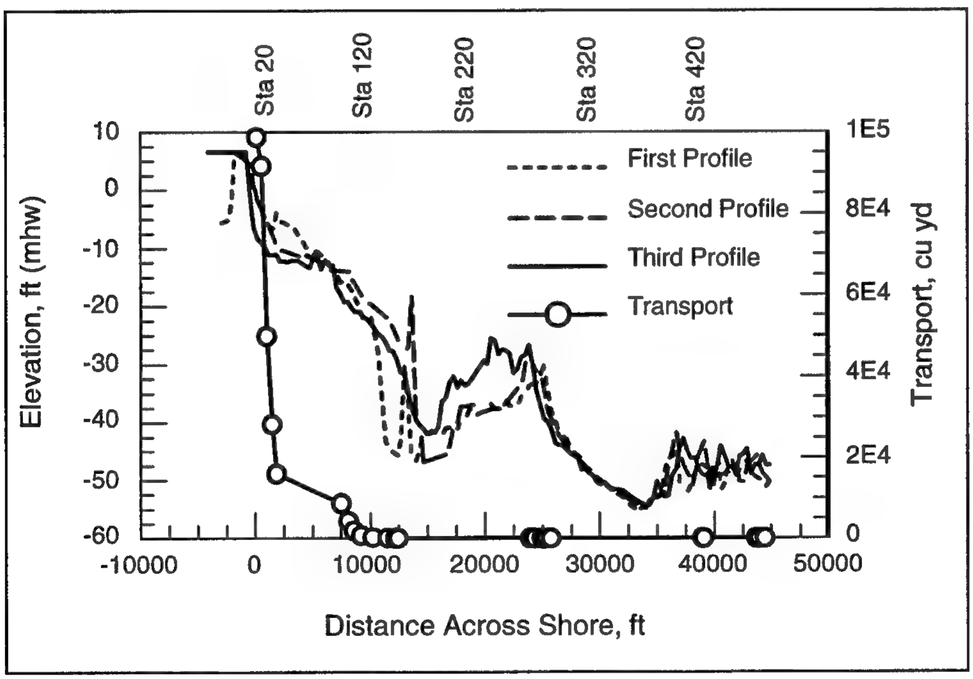

Figure 49. Beach cross sections and southerly longshore transport, 1996

Figure 50 plots the change in southerly longshore transport as a function of distance from the shore. The longshore transport decreases sharply between the shoreline and 2,000 ft offshore. Sta 40 is about 2,000 ft offshore. The north jetty is about 13,000 ft long, so most of the southerly transport along Cumberland Island takes place along the western end of the jetty. Sand moving through the jetty does not necessarily travel directly south and into the channel. It can be moved to the east and west by the tidal current, mostly to the east because the ebb current is dominant. Therefore, sand transported to the channel by the longshore current may act as a sediment supply to areas further east.

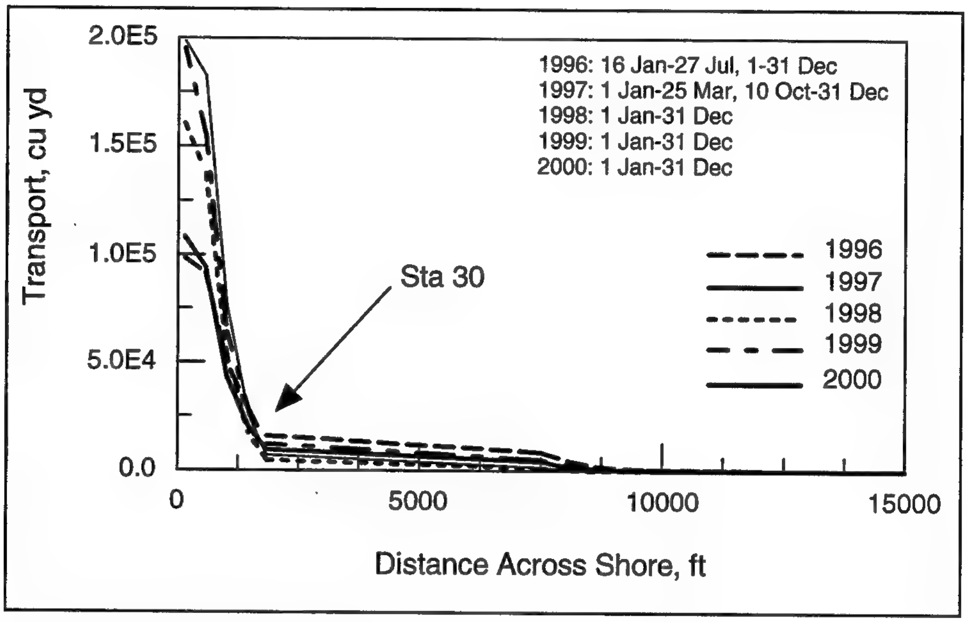

Figure 50. Distribution of longshore transport across the shore

4 Circulation Analysis

Overview of Procedure

Measurements of the current at St. Marys Entrance could not be found for analysis in this study. Therefore, to interpret sediment transport paths inferred from DMS procedures discussed in Chapter 2, as well as to evaluate alternatives for entrance channel maintenance, a tidal circulation model was established for the study area. The tidal current at St. Marys Entrance was simulated with ADCIRC (Luettich, Westerink, and Scheffner 1992), a two-dimensional, depth-integrated, finite-element hydrodynamic model. St. Marys Entrance is located within the domain of the community model developed by CHL for northern Florida and Georgia (Figure 51).

For this DMS application, the modeled current was simulated by boundary forcing of the community model grid with tidal harmonics only, omitting wind and wave forcing. The model was driven with seven astronomical tidal constituents obtained from the Eastcoast 2001, which became available during the course of this study. The database of tidal constituents is described by Mukai, Westerink, and Luettich (2002). The overtide constituents (M_4, M_6) were not applied because the model was forced from deep water.

The domain of the ADCIRC model is 621,500 sq km, and it has 15,159 nodes. The largest element is 166 sq km, and the smallest is 413 sq m. Model resolution at the study site is depicted in Figure 52. Substantial effort was made to represent St. Marys Entrance with high resolution. In particular, the north jetty crest elevation was placed at mean tide level (mtl) and the south crest was placed at 2.5 ft mtl[1] so that during higher tide, water can flow over the structure.

Calibration

The circulation model was verified by comparison to NOS water elevation measurements. Data from four tide stations (Table 9, Figure 53) were selected. The water elevations predicted by the circulation model were compared to water elevations measured at the tide stations for 30 days starting on 29 May 2000. Figures 54-57 show the calculated and measured water levels at the four stations.

[1] Jetty crest elevation was specified after consultation with Mr. Tom R. Martin, Coastal and Hydraulics Branch, Engineering Division, Jacksonville District. The elevation reported from 1927 for the jetties was 5.9 ft mlw and 6.9 ft mlw for the north and south jetty, respectively. Because the jetties are permeable and have settled, their elevation was assumed to be lower.

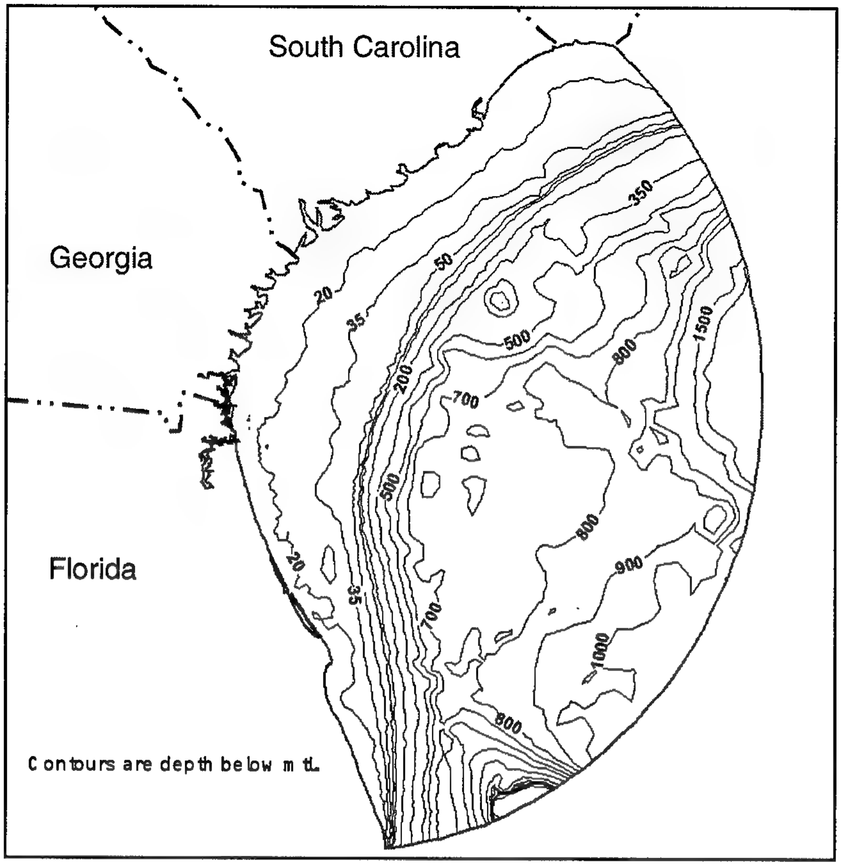

Figure 51. Community model for northern Florida and Georgia

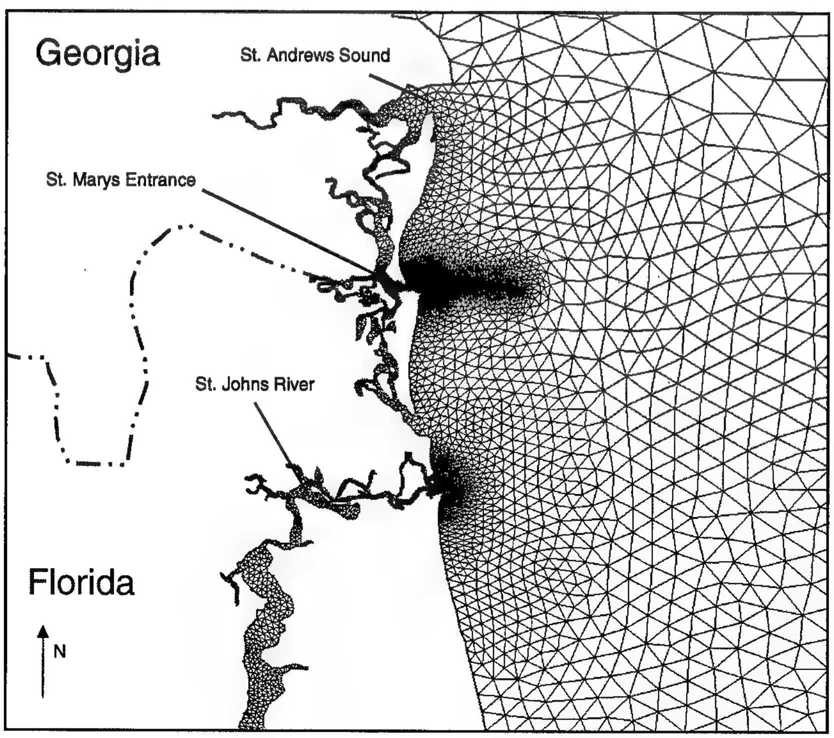

Figure 52. Regional ADCIRC mesh

Table 9
Tidal Datums, m, for NOS Tide Stations (Location and Station Number)

Datum	Fernandina, FL 8720030	Ft. Pulaski, GA 8670870	Mayport, FL 8720220	St. Augustine, FL 8720576
Highest observed water level	4.21	3.40	2.29	2.24
mhhw	2.01	2.28	1.50	1.54
mhw	1.91	2.16	1.42	1.44
mtl	0.98	1.12	0.73	0.75
mlw	0.06	0.07	0.05	0.05
mllw	0.00	0.00	0.00	0.00
Lowest observed water level	-1.17	-1.33	-0.98	-0.99

Note: mhhw = mean higher high water, mhw = mean high water, mtl = mean tide level, mlw = mean low water, mllw = mean lower low water. Datum information obtained from NOS website at http://co-ops.nos.noaa.gov/bench.html

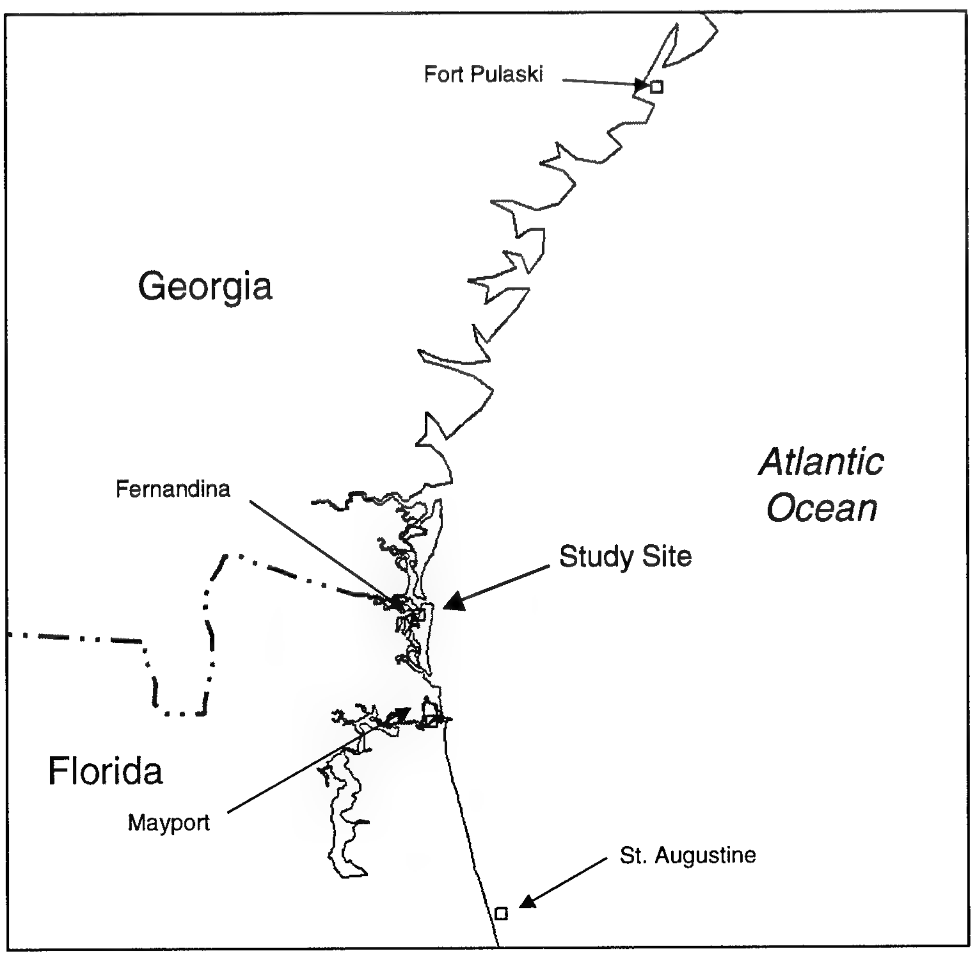

Figure 53. Location of tide stations

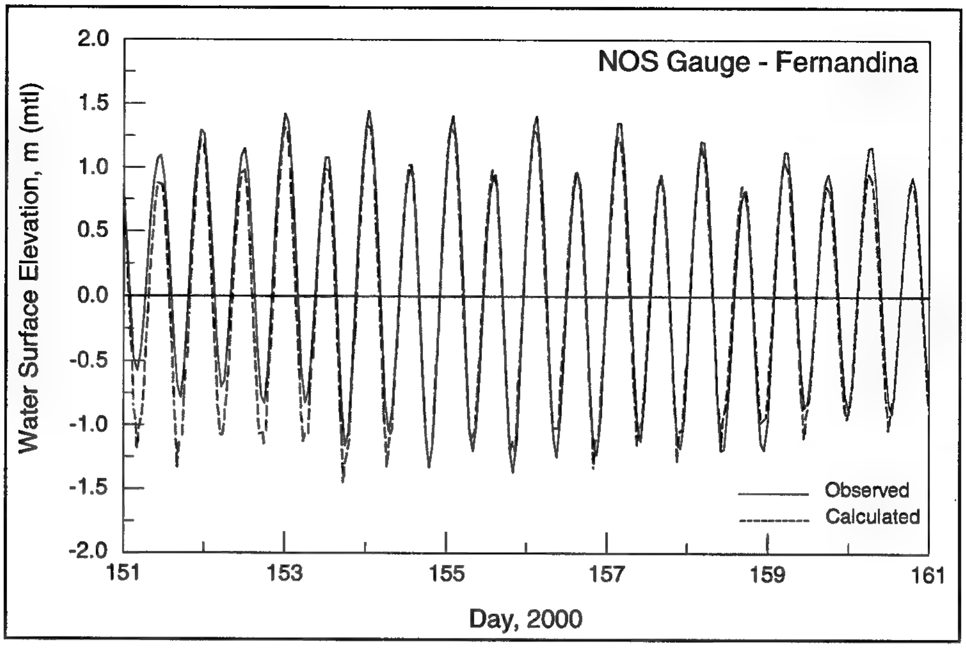

Figure 54. Comparison of measured and calculated water level at Fernandina

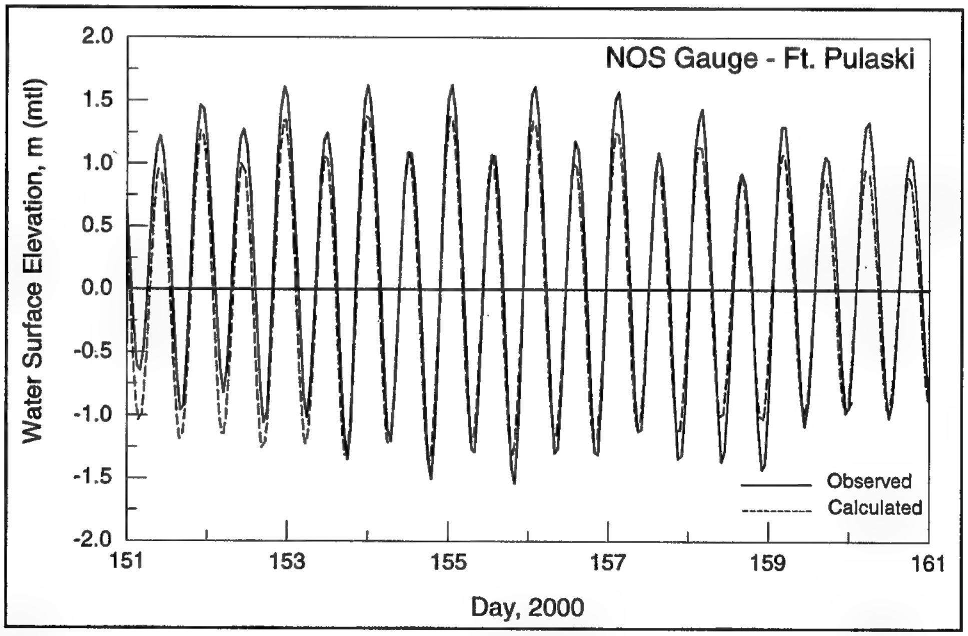

Figure 55. Comparison of measured and calculated water level at Ft. Pulaski

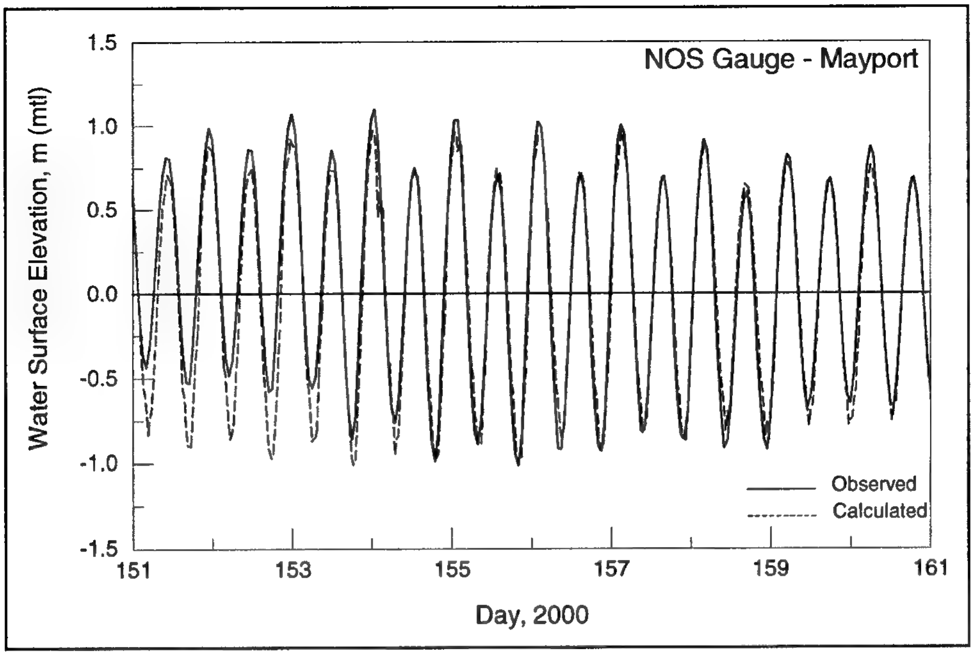

Figure 56. Comparison of measured and calculated water level at Mayport

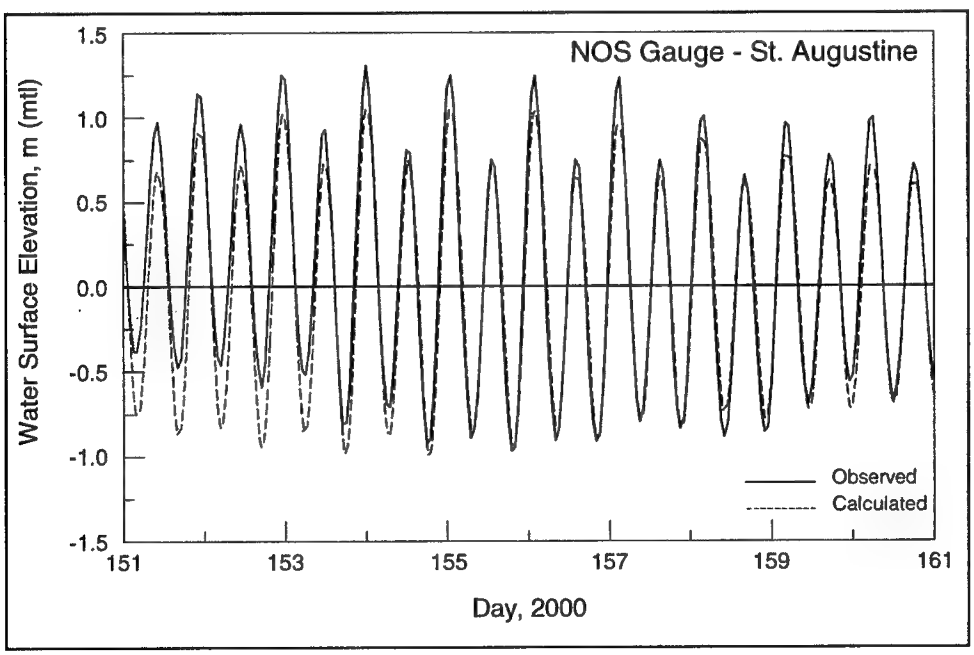

Figure 57. Comparison of measured and calculated water level at St. Augustine

The average root mean square (rms) error was 0.12 m, and the average percent error was 6.4 percent (Table 10). The smallest error, 5.2 percent, was at the Fernandina station, which is within the study area. Some of the discrepancy in the comparison is likely associated with meteorological forcing (wind, air-pressure gradients) not represented in the tidal constituents. The good comparison is attributed to the availability of accurate bathymetry data and rigorous boundary forcing as furnished by the community model.

Table 10
Comparison of Measured and Calculated Water Level

Station	rms Error, m	Percent Error
Fernandina, FL, 8720030	0.10	5.2
Ft. Pulaski, GA, 8670870	0.15	6.6
Mayport, FL, 8720220	0.12	7.2
St. Augustine, FL, 8720576	0.10	6.7

Existing Condition

Calculated speeds for the tidal current range from 0 to 1 m/sec (Figures 58 and 59). It is known empirically that a stable inlet will have maximum current velocity on the order of 1 m/sec (O'Brien 1966). The calculations therefore indicate that the inlet entrance has a stable cross-sectional area and is not expected to change greatly. Because on average the cross-sectional area will remain the same, sediment infiltrating the inlet will tend to be swept either to the flood shoal or to the ebb shoal. The objective of the present study is to optimize the distribution of the sediment shoaling to reduce channel maintenance costs.

The magnitude of the peak flood current is slightly less than that of the peak ebb current, indicating that the net sediment transport in the channel should be to the east, toward the ebb shoal. More of the flood current enters the channel through or over the jetties than exits through or over the jetties on the ebb tide because the peak ebb current occurs while the jetty crest is above the surface of the water, whereas the peak flood current occurs while the north jetty is submerged. At peak ebb flow, the water surface is at -0.67 m mtl, which is below the crest of the north and south jetty (Figure 60). The water surface is at 0.08 m mtl at peak flood tide. At peak flood tide the north jetty (0 m mtl) is submerged, and the south jetty (0.6 m mtl) is exposed. The flood tidal current causes the sand waves located just south of the north jetty to migrate into the channel. Sand is carried over or through the north jetty, to the southwest, during the flood tide. Once the tide turns, the ebb current tends to move the sediment eastward (see Figure 25).

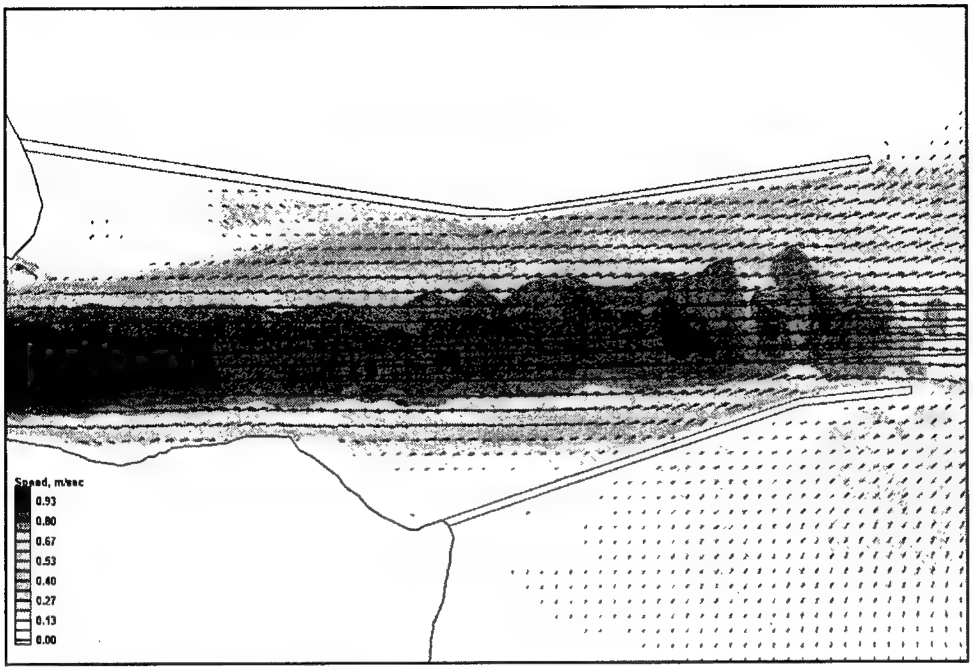

Figure 58. Peak ebb current

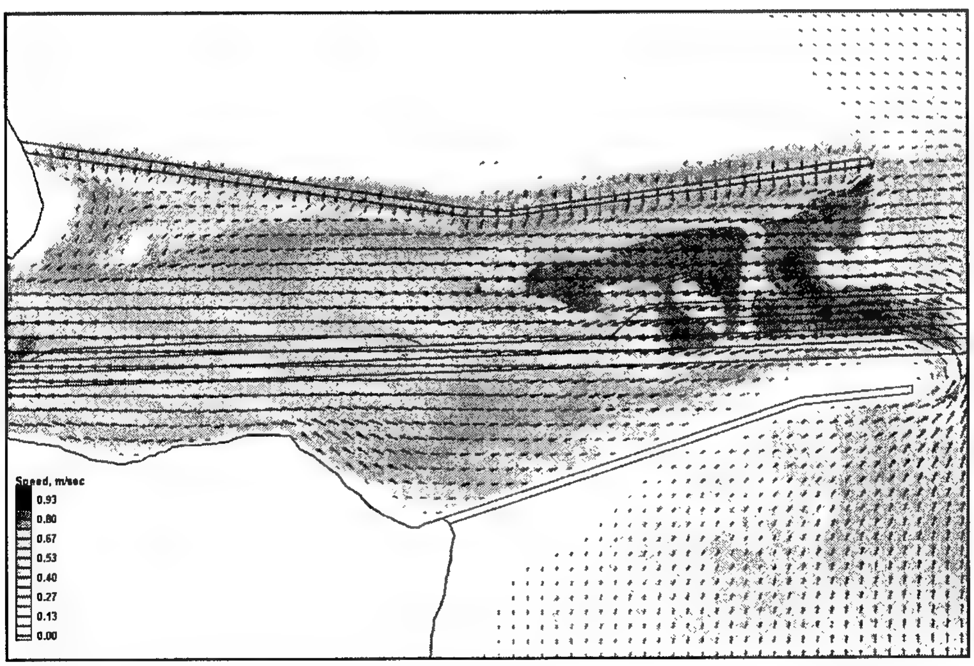

Figure 59. Peak flood current

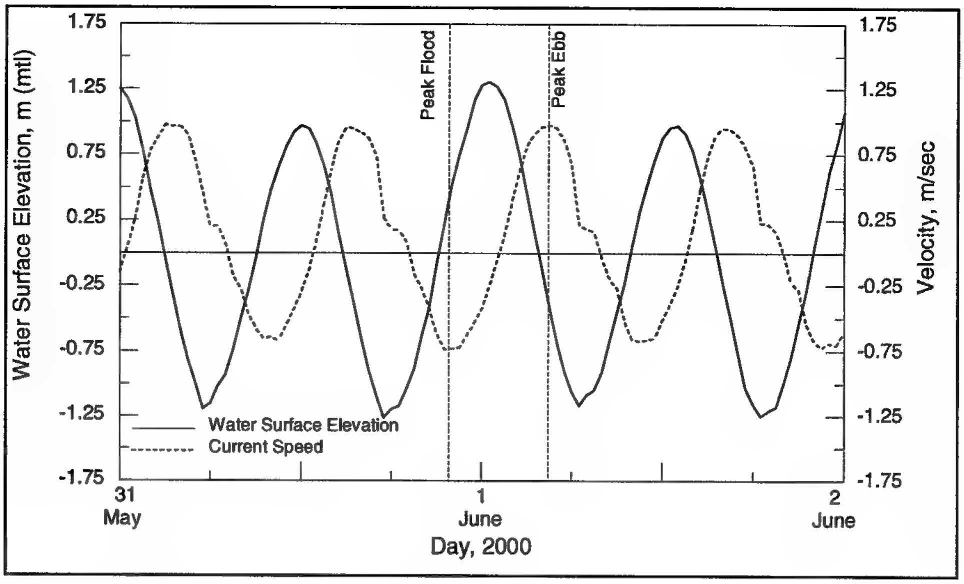

Figure 60. Tidal current and water elevation

During both the peak ebb and flood tides the current velocities approach zero east of the continental shelf (Figures 61 and 62). In the south section of the circulation domain, the tidal currents increase in strength because of the shallow water near the Bahamas. The community model performs well as the tidal wave is propagated into enclosed estuarine environments and predicts stronger current in these areas (Figures 63 and 64).

Figures 65 and 66 show the peak flood and ebb currents, respectively. The tidal wave approaches the entrance from the southeast (Figure 65). The flood current direction is more northward closer to the shore. This provides more sediment to the channel through longshore transport from the south. Figure 66 shows an eddy formed by the ebb jet as it exits the inlet. This eddy will also provide sediment to the channel from the south. A section of the Atlantic Intracoastal Waterway (AIWW) (shown in Figure 67) was removed from the modeling domain so that the time-step of the ADCIRC model could be increased for several production runs.

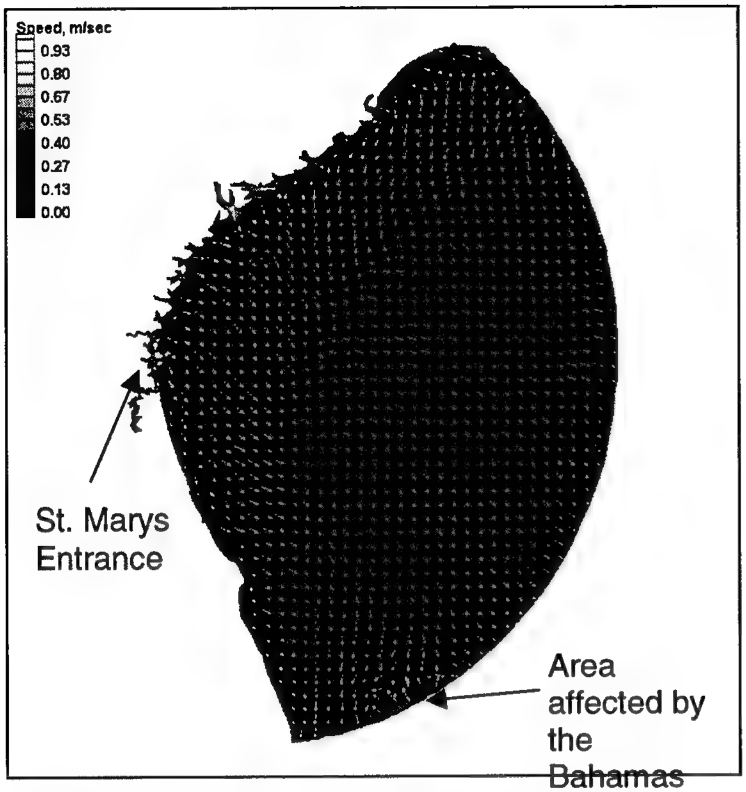

Figure 61. Current speed at peak ebb, domain scale

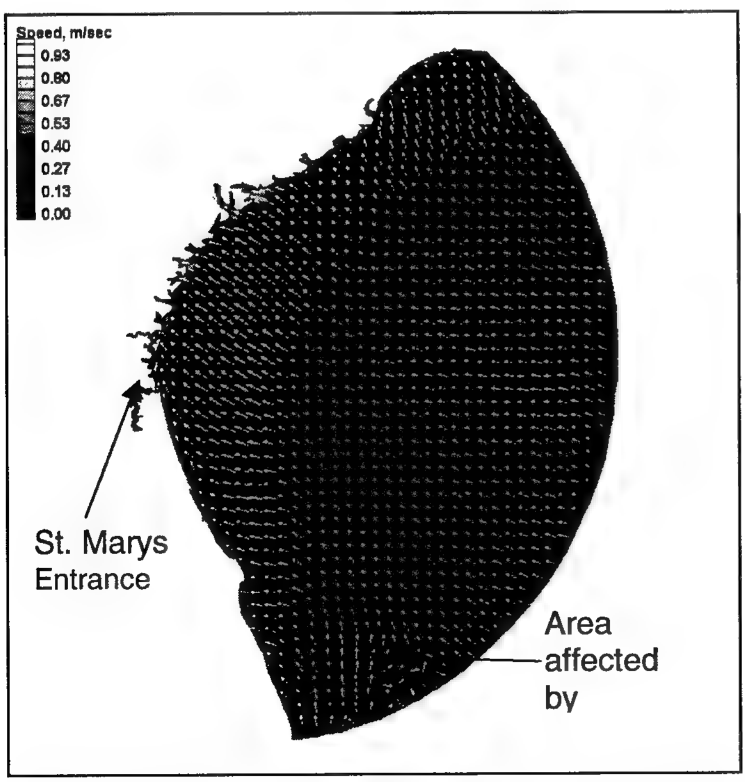

Figure 62. Current speed at peak flood, domain scale

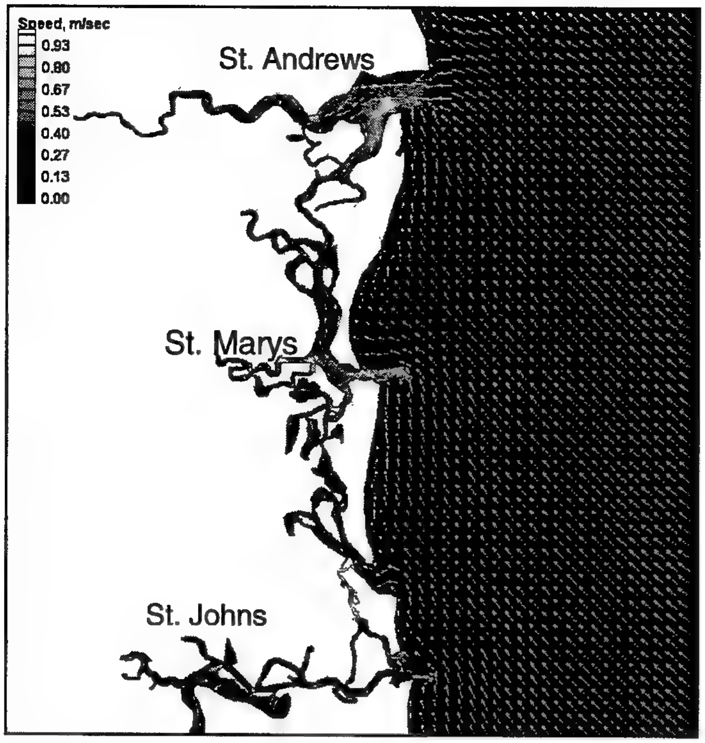

Figure 63. Current speed at peak flood, estuary scale

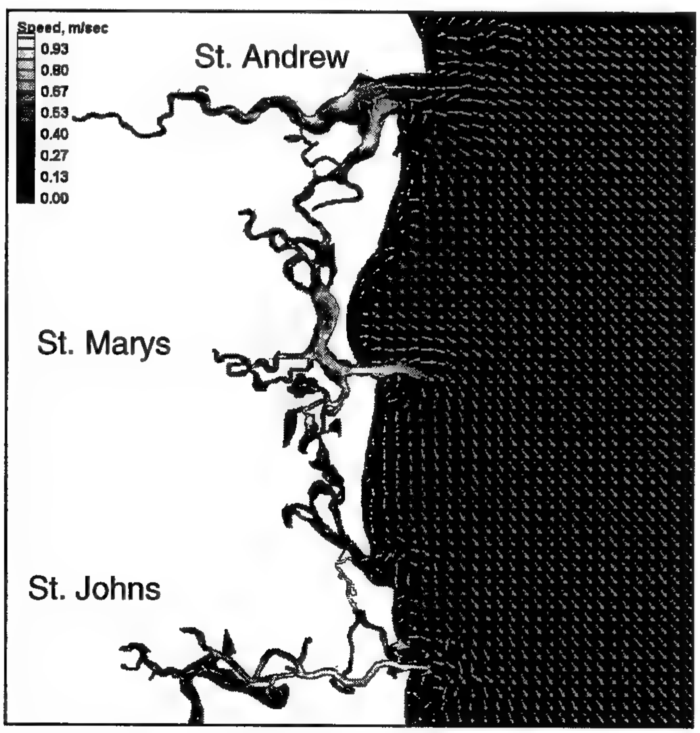

Figure 64. Current speed at peak ebb, estuary scale

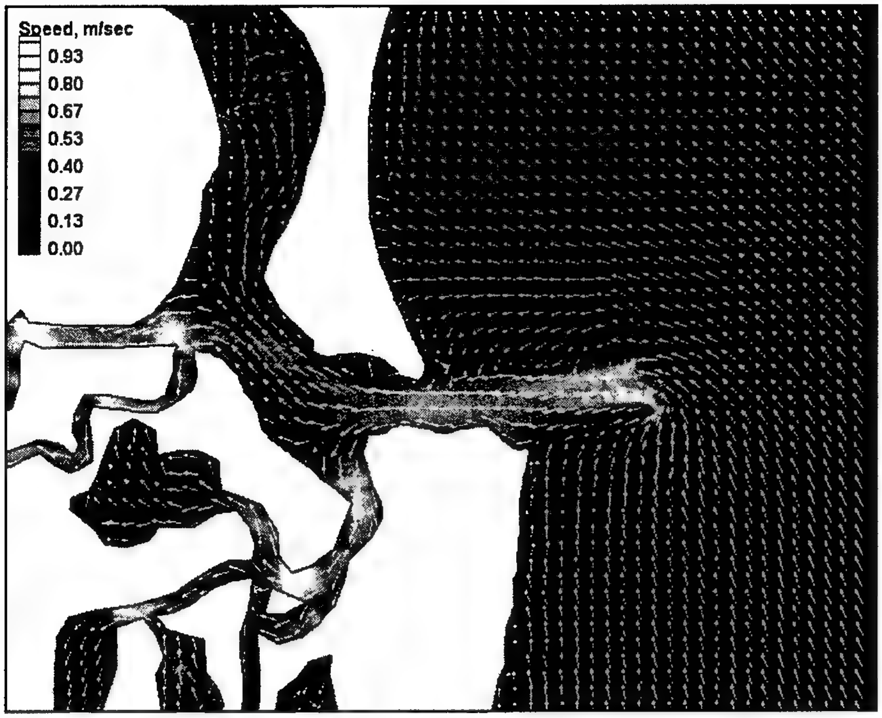

Figure 65. Current speed at peak flood, project scale

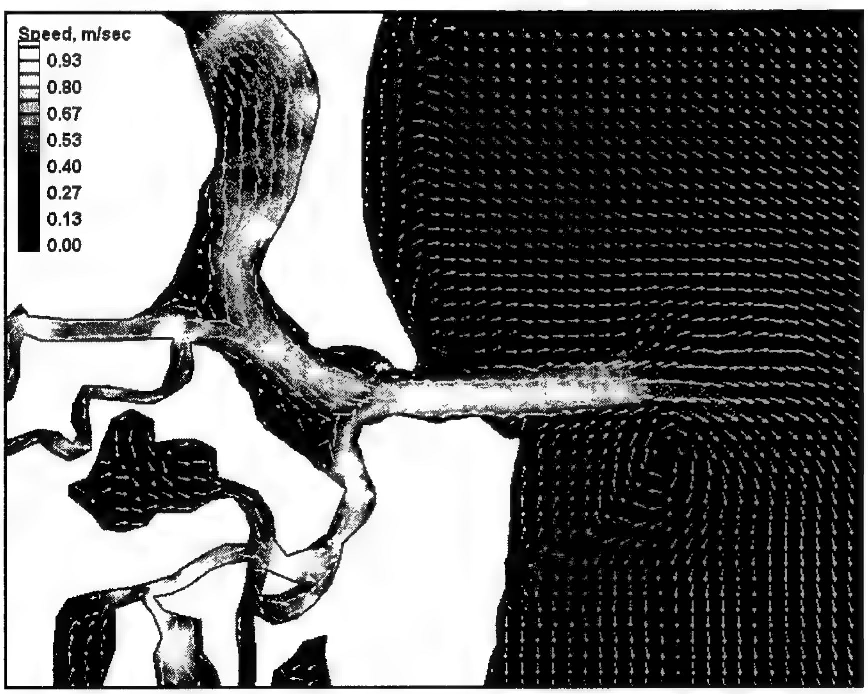

Figure 66. Current speed at peak ebb, project scale

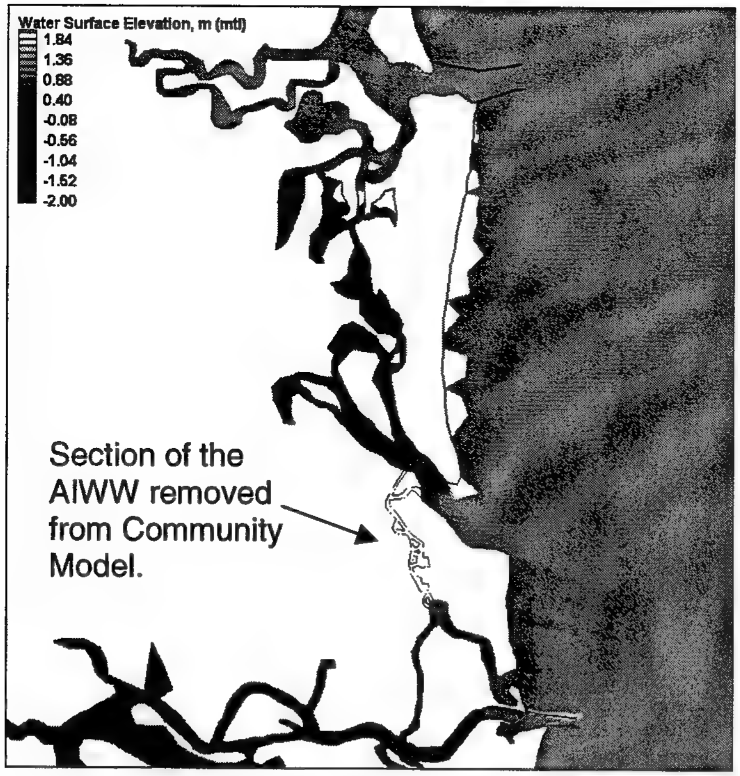

Figure 67. Water surface elevation, project scale

5 Evaluation of Alternatives

This chapter synthesizes findings from the preceding chapters in developing and evaluating alternatives to reduce the frequency of maintenance of the entrance channel. The alternatives, listed in Table 11, were developed by consideration of past channel and widener performance, and inferences about sediment paths. Alternatives are considered for two major shoaling areas, the inner (sta 100-225) and outer channel (sta 225-340).

Table 11
Definition of Alternatives

Alternative	Description
Inner Channel, sta 100-225	
1	Extend existing north channel widener 1,500 ft to the west, to sta 105
2	Extend existing north channel widener 1,500 ft to the west and 300 ft to the north
3	Sand tighten south jetty
4	Sand tighten north jetty
Outer Channel, sta 225-340	
5	Advance dredging

Alternatives for Inner Channel, Sta 100-225

Alternatives 1 and 2: Channel wideners

In 1987, wideners were added to the channel. The volume analysis described in Chapter 2 determined that the wideners are functioning as planned and are decreasing shoaling within the channel. The wideners serve as deposition basins to intercept and collect sand approaching the channel from the north or south before it enters the authorized channel. Some sand approaching the entrance will come over or through the jetty. The location of the northern widener allows it to capture a portion of that sand before it reaches the navigation channel. On average, it is estimated that the north and south wideners together decrease the need for channel maintenance by 2.5 months to 5 years, dependent on the wave conditions and antecedent state of the inlet morphology (Table 4).

Here, two alternative configurations of channel widener are examined to determine if they would be more effective than the existing design. Changes in the footprint of the wideners and the associated changes in the horizontal pattern of the tidal current at the western corner of the northern channel widener are of most interest because this area shoals more than the remainder of the channel.

Two alternatives were developed for investigating possible reduction in dredging frequency in the inner channel through modification of the wideners. Alternative 1 extends the existing channel widener to sta 105, about 1,500 ft to the west, and Alternative 2 extends the existing channel widener 1,500 ft to the west and 300 ft to the north (Figure 68). It is assumed that both alternatives would be maintained to the same depth as the channel.

The circulation model was run with configurations for Alternatives 1 and 2. The channel and the corresponding widener were dredged to a minimum depth of 51 ft mllw as an initial condition. Alternative 1 covers 422,000 sq ft and would require 174,000 cu yd of new work to implement the alternative, based on the 1979 survey. Alternative 2 covers 3,927,000 sq ft and would require 2,151,000 cu yd of new work to implement based on the 1979 survey. A new bathymetric survey of the subject areas would need to be done to determine an updated volume necessary to be dredged.

Calculated current velocities for the existing condition were compared to the velocities calculated for Alternative 1 and 2. The peak ebb and flood velocities for the alternatives are shown in Figures 69 to 72. Both alternatives for ebb and flood tides showed a decrease in velocity just west of the existing widener. The local velocity decreases because this area was deepened to represent each alternative condition. Alternative 2 has a greater decrease in velocity, because more area has been deepened. More detailed analysis of velocity changes did not indicate a clear improvement in channel performance with either alternative based on change in the tidal current, which was relatively minor (typically less than 0.05m/sec at peak ebb or flood).

The most significant shoaling along the channel occurs near channel marker R-22, or sta 120 (Figures 23-25), caused by intrusion of a shoal. The speed of this shoal migration was estimated from changes in the -45 ft mllw contour, with distance measured along a southeast orientation. The shoal migrated between 210 and 530 ft/year to the southeast. The additional dredging required for Alternative 1 would further remove the migrating shoal from the navigation channel. Figure 73 shows Alternative 1 with the conservative and minimal annual shoal migration. The outer circular area shows the conservative distance, and the inner grid pattern shows the minimal distance. To delay the shoal from entering the navigation channel for 1 year the adapted alternative should extend beyond the conservative shoal migration zone. Given that the shoal enters the channel east of channel marker R-22, Alternative 1 appears to provide the required setback.

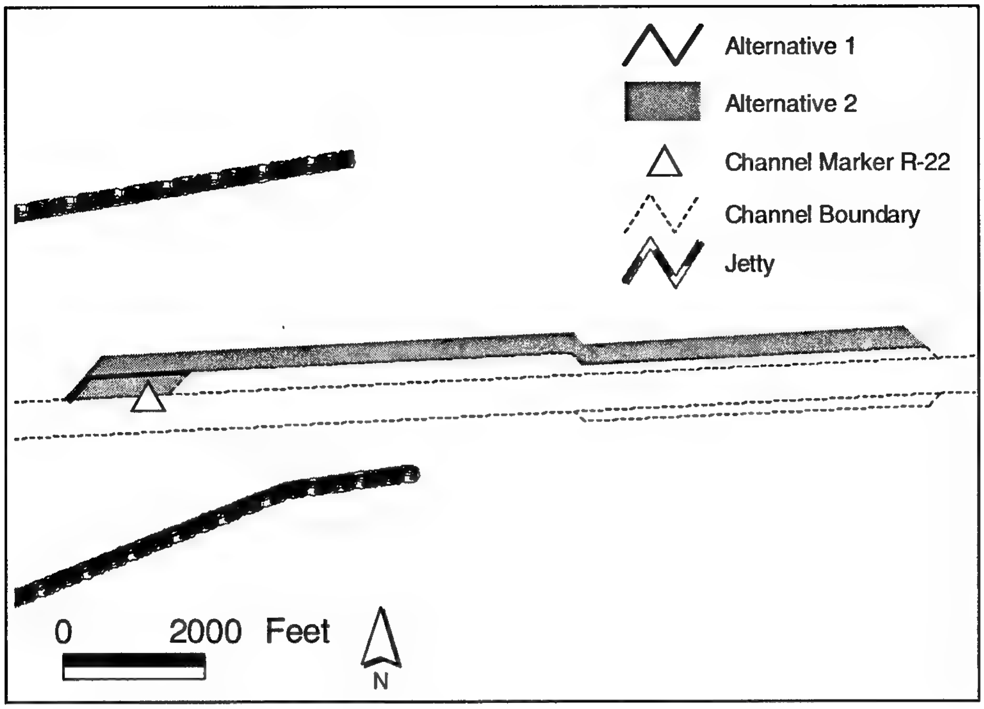

Figure 68. Alternative channel designs

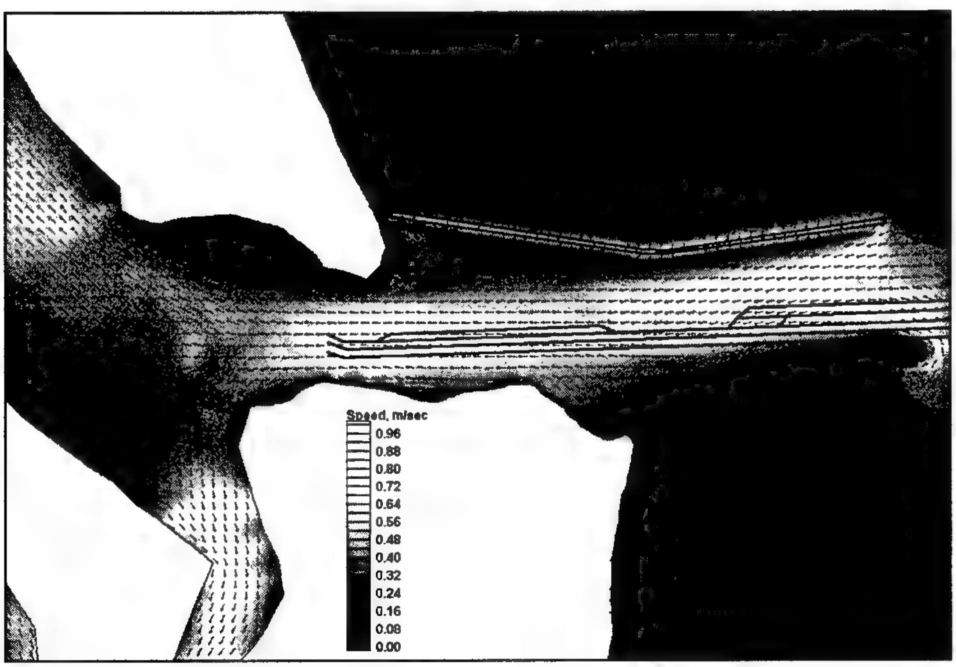

Figure 69. Peak flood, Alternative 1

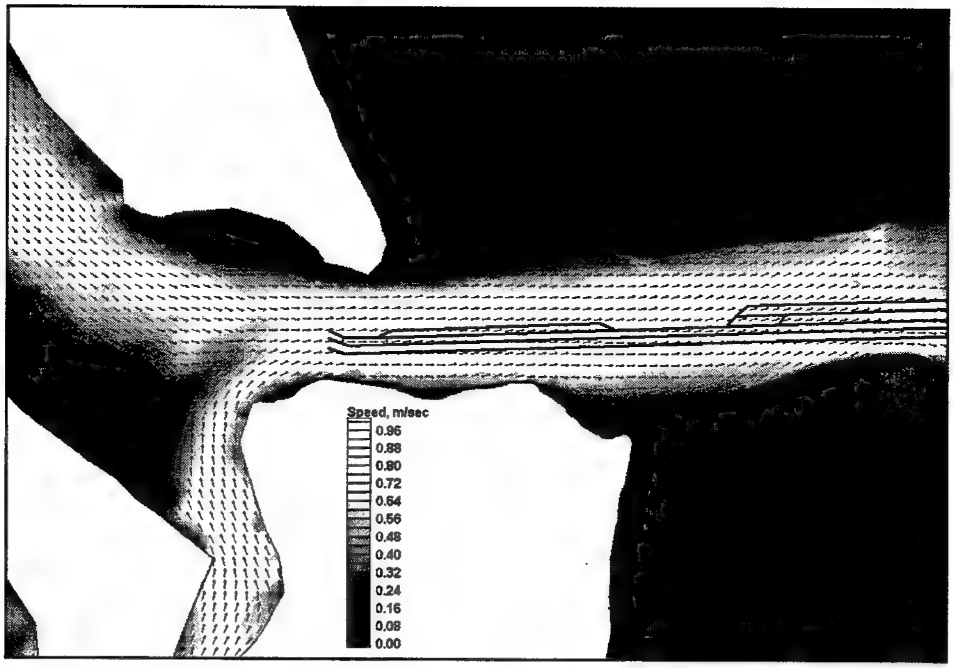

Figure 70. Peak ebb, Alternative 1

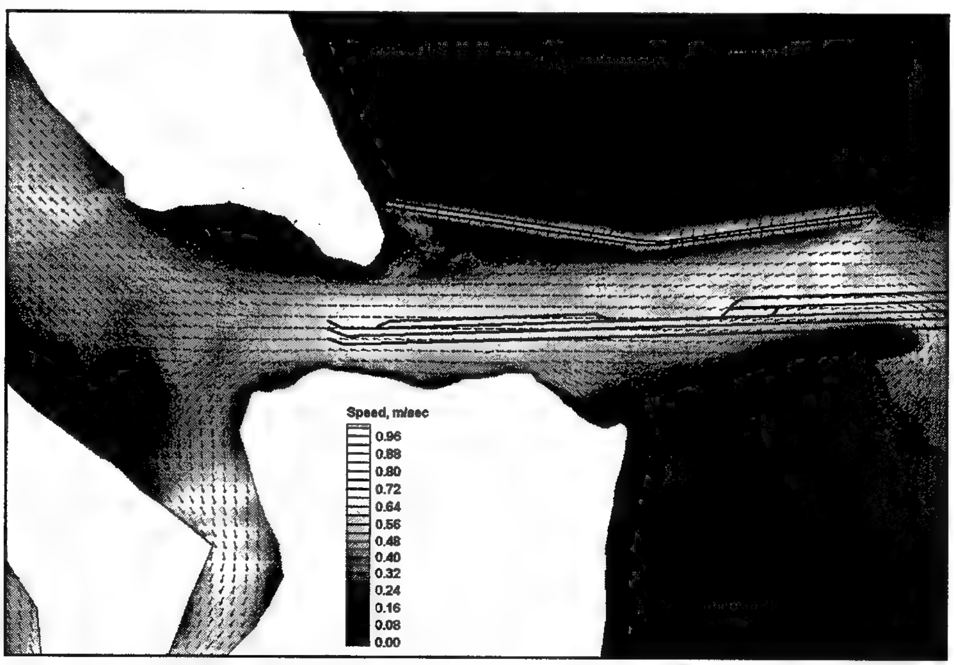

Figure 71. Peak flood, Alternative 2

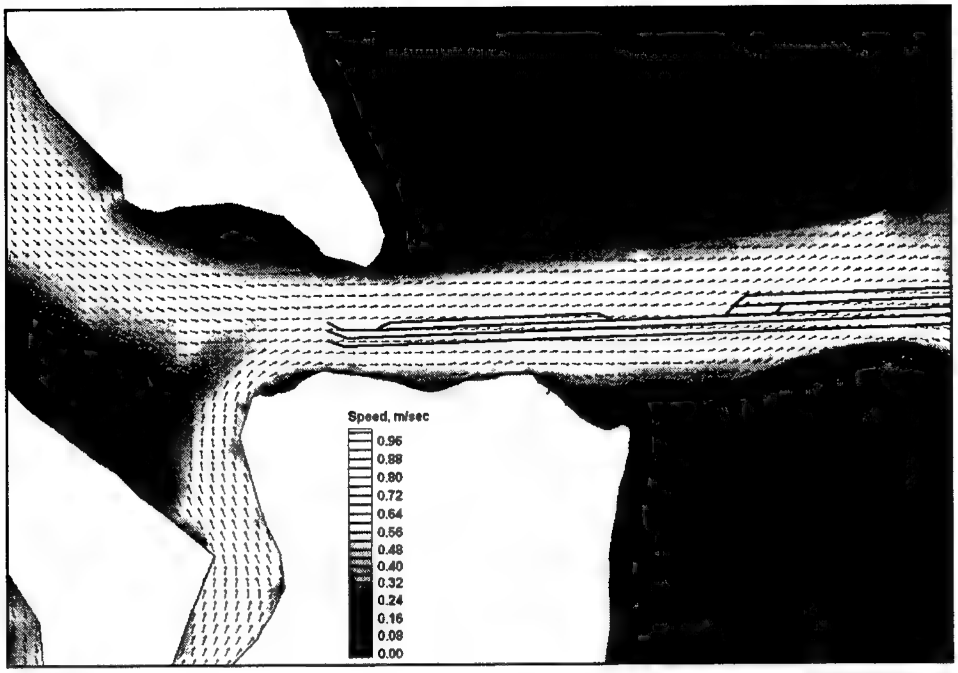

Figure 72. Peak ebb, Alternative 2

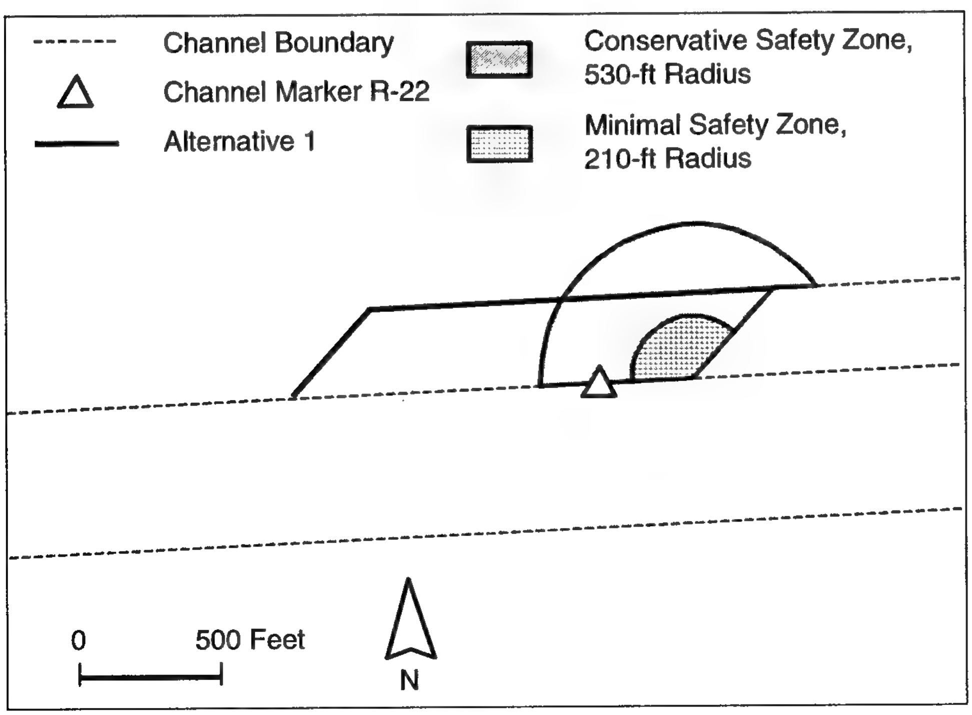

Figure 73. Alternative 1, conservative and minimal shoal migration

Alternative 3: Sand tightening south jetty

In 1987, the Jacksonville District sand tightened the landward 1,500-ft-long segment of the south jetty. This action prevents a portion of the sand moving north along Amelia Island from entering the navigation channel, but 34 percent is still estimated to move though the jetty between the shoreline and the sand-tightened section (Raichle, Bodge, and Olsen 1997). Raichle, Bodge, and Olsen (1997) also estimated that 597,000 cu yd/year (311,000 cu yd/year is north longshore transport and 286,000 cu yd/year is from erosion of Amelia Island) approaches the channel from the south. The sand supply and sediment path assumed by Raichle, Bodge, and Olsen (1997) would mean that 203,000 cu yd/year of sand may enter the channel. In contrast to these values for transport across the south jetty, Grosskopf and Kraus (1994) calculated the north-directed transport south of the channel to be a smaller amount, 150,000 cu yd/year.

Considering the variations in estimates of sand entering the channel through the jetty, extending the previous sand tightening project west to the dune has the potential to stop 150,000 to 203,000 cu yd/year from entering the channel. Preventing sand from entering the channel by sand tightening would promote beach accretion on north Amelia Island. However, this alternative would also reduce the sand available to the interior of Amelia Island, Fort Clinch, so if this option is selected it should be combined with monitoring and renourishment of the Fort Clinch area.

Alternative 4: Sand tightening north jetty

Because partial sand tightening of the south jetty reduced the amount of material reaching the channel from Amelia Island, tightening the north jetty was considered in this study. Raichle, Bodge, and Olsen (1997) estimated that 195,000 cu yd/year entered through the north jetty. The analysis of the cross-shore longshore sediment transport distribution done in the present study shows that most of the sand carried with the longshore current is located within 2,000 ft of the beach (Figures 48-50). Sand tightening the first 2,000 ft of the jetty would prevent about 91,000 cu yd/year from entering the channel (Raichle, Bodge, and Olsen 1997). Although sand tightening of the north jetty would decrease the amount of material deposited into the channel and advance the southern beach of Cumberland Island, it would not improve the condition of the beaches on Amelia Island. Also, after the beach adjusts to the new sand-tightened jetty, the impounded sand would extend the shoreline seaward until the sand was once again able to move through the jetty and enter the channel.

Alternative for Outer Channel, sta 225-340, Alternative 5: Advance Dredging

Mitigation options for shoaling in the outer channel are limited because this area is in open water and sediment is supplied along-channel. This area is located more that 4 miles offshore, in 40 to 50 ft (mllw) water depth. Therefore, advance dredging is recommended to reduce mobilization costs and take

advantage of favorable weather and environmental windows. This section discusses the effectiveness of advance dredging.

The average shoaling rate between sta 225 and 340 is 12,000 cu yd/year per 200 ft of channel or 3.0 ft/year[1] over the channel width of 500 ft. One additional foot of advance dredging could delay the need for dredging by 4 months, on average. Because channel maintenance is typically done annually, it may be beneficial to advance dredge by 3 ft to extend the time between maintenance to a year. Presently, the Jacksonville District is advance dredging by 2 ft. Adding 3 ft of advance dredging may decrease the need for maintenance by an additional year, but would require 639,000 cu yd more to be dredged each time.

These estimated maintenance requirements are based on the average past channel conditions and assume uniform filling of the channel. This average will not represent the situation in the future; therefore, the actual maintenance requirements will vary from these estimates depending on environmental conditions. Review of past conditions shows that there are significant variations from the average. Shoaling rates of 5,000 to 20,000 cu yd/year/200 ft or 1.5 to 5.5 ft/year are common between sta 225 and 340. Considering the range of shoaling rates, based on historical performance, advance dredging of 3 ft could extend the time between dredging events anywhere between 6 to 22 months, depending on the sediment transport rates at the particular time. In particular, the frequency and severity of storms may control shoaling rates and, therefore, the time between required channel maintenance. More storms would increase the shoaling rate and decrease the allowable time between maintenance. Also, as the side slopes of the deepened channel readjust, the channel may temporarily experience greater shoaling rates.

[1] Values are rounded to the nearest half-foot.

6 Conclusions and Recommendations

Bathymetric surveys, dredging records, and the published literature were reviewed in this study. Wave modeling was conducted to estimate the magnitude and direction of the wave-induced longshore sand transport. A project-specific tidal circulation model was established within the existing community model grid and run for the existing bathymetry and two alternatives for the channel wideners.

This study, following the DMS methodology, identified two main causes of shoaling at St. Marys Entrance. Figure 74 is a summary concept diagram of the shoaling areas, shown as circles, and their sediment sources, shown as arrows. In the inner channel shoaling area, sta 100-225, sand is supplied from the north and south by longshore transport. The primary pathway into the channel is by sand wave migration. In the outer channel shoaling area, sta 225-340, sediment is transported from the east and settles out of the water column as the ebb current weakens near the edge of the ebb shoal.

The following summarizes the recommended maintenance alternatives discussed in Chapter 5:

a. Inner channel, sta 100-225: Alternative 1, extend north widener west to sta 105.

b. Outer channel, sta 225-340: Advance dredge by 3 ft.

Inner Channel Shoaling

Inner channel shoaling, defined to correspond to the channel segment between sta 100-225, is caused primarily by sand transported into the channel from the north and south by longshore transport. The sand may enter through, over, or around the jetties. North of the navigation channel is a sand wave that migrates into the channel and enhances shoaling near channel marker R-22, sta 120. Of the four alternatives considered for the inner channel, Alternative 1 is recommended to decrease shoaling and maintenance cost in the inner channel because it focuses on the area near sta 105.

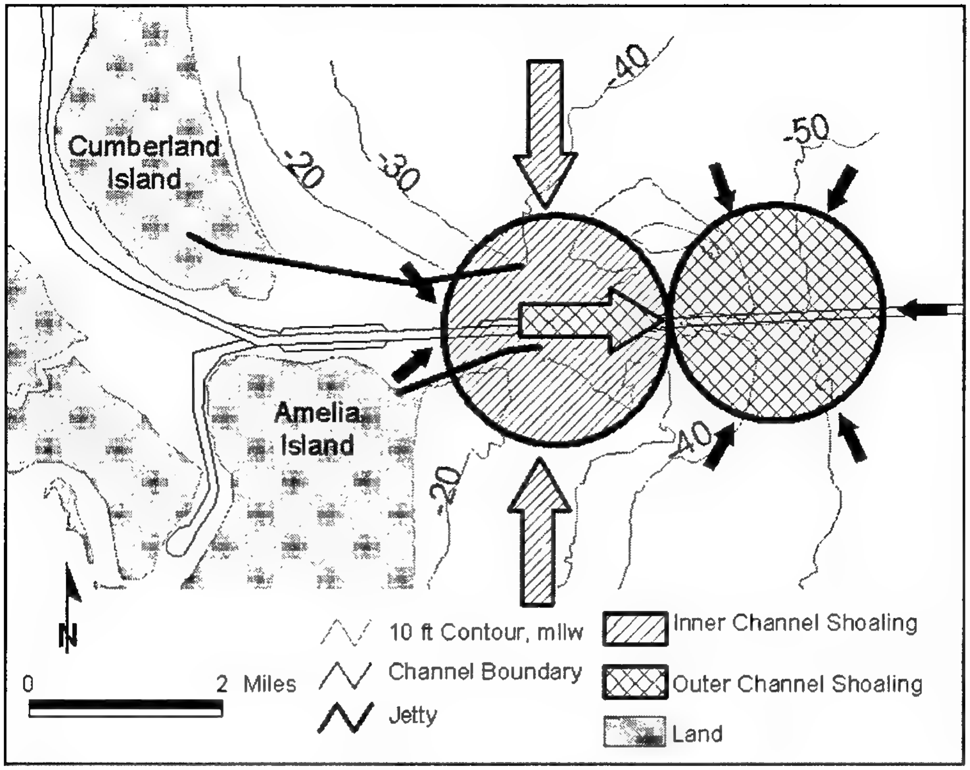

Figure 74. Diagram of shoaling areas and sediment sources

Analysis showed that the presently implemented design of the channel widener functions successfully in decreasing channel shoaling and maintenance by capturing sand from the north and the south of the channel. Alternative configurations of the channel wideners are presented in Chapter 5 to determine if the channel configuration near sta 120, channel marker R-22, was contributing to the chronic shoaling in this area. Analysis of alternative channel configurations and historical bathymetry concluded that the channel configuration does not contribute to the channel shoaling near sta 100 or channel R-22.

Examination of the simulated tidal currents in the area and the 1979 bathymetry indicates the tidal current causes this shoal to migrate into the channel. This process is independent of the configuration of the channel (see Figure 25). Alternative 1 provide an additional set back from the navigation channel for this shoal and could reduce the need for unplanned dredging such as was required in October 1994. Extension of the north widener east to sta 105 (Figure 73) would require 174,000 cu yd of new dredging and should extend the maintenance interval by at least 1 year. This estimate is based on the past environmental conditions and will change if the wave conditions change. For example, if more severe storms pass the Entrance, then the shoal would be expected to migrate faster, and additional dredging may be necessary within a year.

Outer Channel Shoaling

Outer channel shoaling occurs past the ebb shoal between sta 225 and 340. Here, the ebb current weakens and sediment in the water column tends to be deposited. The sediment supply to the outer channel cannot be decreased because that would involve obstructing the navigation channel. Therefore, advance dredging appears to be the only mitigation option to address shoaling by the fine-grained material collecting there. Advance dredging of 3 ft is expected to extend the need for channel maintenance by at least 7 months, although variation in this value can be expected, as discussed in Chapter 2.

References

Aubrey, D. G., McSherry, T. R., and Spencer W. D. (1991). "Sedimentation study environmental monitoring and operation guidance system (EMOGS), Kings Bay, Georgia and Florida, 1988-1990, Final Report," WHOI-91-17, (CRC-91-01), Woods Hole Oceanographic Institution Technical Report, 96 pp.

Craig, K. R., Gosselin, M. S., Cook, D. S., and Pratt, T. P. (2001). "DMS: Diagnostic Modeling System; Report 3, The DMS Data Manager-A user's guide," Technical Report TR-99-19, Coastal and Hydraulics Laboratory, U.S. Army Engineer Research and Development Center, Vicksburg, MS.

Dean, R. G. (1988). "Sediment interaction at modified coastal inlets: Processes and policies," *Lecture Notes on Coastal and Estuarine Studies 29, Hydrodynamics of Tidal Inlets*, D. G. Aubrey, and L. Weishar, ed., Springer-Verlag, New York, 412-439.

Florida Coastal Engineers, Inc. (1976). "Beach erosion control study Nassau County, Florida," sponsored by U.S. Army Engineer District, Jacksonville.

Gorman, L. T. (1991). "Assessment of the nearshore zone at St. Marys Inlet, Florida," *Proceedings Coastal Zone '91,* American Society of Civil Engineers, New York, 607-617.

Gorman, L. T., and Pope, J. (1994). "Regional, physical, and engineering setting," *Kings Bay Coastal and Estuarine Physical Monitoring and Evaluation Program; Coastal studies, Volume 1: Main text and Appendix A*," Technical Report CERC-94-9, N. C. Kraus, L. T. Gorman, and J. Pope, ed., U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Gorman, L. T., Pope, J., and Pitchford, K. R. (1994). "Profile surveys, sediments, and beach fills," *Kings Bay Coastal and Estuarine Physical Monitoring and Evaluation Program: Coastal studies, Volume 1: Main text and Appendix A*, Technical Report CERC-94-9, N. C. Kraus, L. T. Gorman, and J. Pope, ed., U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Griffin, M. M., and Henry, V. J. (1984). "Historical changes in the mean high water shoreline of Georgia 1857-1982," Bulletin 98, Georgia Geological Survey, Atlanta, GA.

Grosskopf, W. G., and Kraus, N. C. (1994). "Shoreline change extrapolation," *Kings Bay Coastal and Estuarine Physical Monitoring and Evaluation Program: Coastal studies, Volume 1: Main text and Appendix A,* Technical Report CERC-94-9, N. C. Kraus, L. T. Gorman, and J. Pope, ed., U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Grosskopf, W. G., Kraus, N. C., Militello, A., and Bocamazo, L. M. (2002). "Implementation of a regional wave measurement and modeling system, south shore of Long Island, New York," *Proceedings Waves '01*, American Society of Civil Engineers, Reston, VA (in preparation).

Headquarters, U.S. Army Corps of Engineers. (1993). "Requirements and procedures for referencing coastal navigation projects to mean lower low water datum," Engineering Technical Letter 1110-1-152, Washington, DC.

Kraus, N. C. (2000). "Introduction to the Diagnostic Modeling System (DMS)," Coastal Engineering Technical Note CETN-IV-28, U.S. Army Engineer Research and Development Center, Vicksburg, MS. http://chl.wes.army.mil/library/publications/chetn/#part4

Kraus, N. C., and Harikai, S. (1983). "Numerical model of the shoreline change at Oarai Beach," *Coastal Engineering* 7, 1-28.

Kraus, N. C., and Taylor, R. B. (2000). "The Diagnostic Modeling System — DMS: A new tool for evaluating the causes of channel shoaling," *Proceedings of the Western Dredging Association Twentieth Technical Conference.* Western Dredging Association and Center for Dredging Studies, Texas A&M University, College Station, TX, 371-376.

Kraus, N. C., Faucette, R. C., and Rogan, M. K. (1997). "Water-level analysis for Cumberland Sound, Georgia," Technical Report CHL-97-11, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Kraus, N. C., Gorman L. T., and Pope, J., ed. (1994). "Kings Bay Coastal and Estuarine Physical Monitoring and Evaluation Program: Coastal studies, Volume 1: Main text and Appendix A," Technical Report CERC-94-9, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Kraus, N. C., Johnston, S., Taylor, R. B., Gosselin, M. S., and Craig, K. R. "DMS: Diagnostic Modeling System; Report 6, The DMS manual," (in preparation), Technical Report TR-99-19, U.S. Army Engineer Research and Development Center, Vicksburg, MS.

Luettich, R. A., Westerink, J. J., and Scheffner, N. W. (1992). "ADCIRC: An advanced three-dimensional circulation model for shelves, coasts, and estuaries; Report 1, theory and methodology of ADCIRC-2DDI and ADCIRC-3DL," Technical Report DRP-92-6, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Mukai, A. Y., Westerink, J. J., and Luettich, R. A. (2002). "Guidelines for using Eastcoast 2001 database of tidal constituents within Western North Atlantic Ocean, Gulf of Mexico and Caribbean Sea," Coastal and Hydraulics Engineering Technical Note CHETN-IV-40, U.S. Army Engineer Research and Development Center, Vicksburg, MS. http://chl.wes.army.mil/library/publications/chetn/#part4

O'Brien, M. P. (1966). "Equilibrium flow area of tidal inlets on sandy coasts." *Proceedings 10th Conference on Coastal Engineering*. American Society of Civil Engineers, New York, 676-686.

Olsen, E. J. (1977). "A study of the effect of inlet stabilization at St. Marys Entrance, Florida." *Proceedings Coastal Zone '77*. American Society of Civil Engineers, New York, 311-29.

Olsen, E. J. (2001). "Pictorial development of the Fort Clinch, Florida T-head groin rehabilitation," *Shore & Beach* 69(2), 17-19.

Parchure, T. M. (1982). "St. Marys Entrance glossary of inlets report #11," Report Number 44, Florida Sea Grant College, Gainesville, FL.

Pope, J. (1991). "Ebb delta and shoreline response." *Proceedings Coastal Zone '91*. American Society of Civil Engineers, New York, 643-54.

Pope, J., and Richardson, T. W. (1994). "Introduction," *Kings Bay Coastal and Estuarine Physical Monitoring and Evaluation Program: Coastal studies, Volume I: Main text and Appendix A*, Technical Report CERC-94-9, N. C. Kraus, L. T. Gorman, and J. Pope, ed., U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Raichle, A. W., Bodge, K. R., and Olsen, E. J. (1997). "St. Marys Entrance inlet management study," Olsen Associates, Inc., Jacksonville, FL.

Richards, R. D., and Clausner, J. E. (1988). "Feasibility of sand bypassing systems for reducing maintenance dredging in the St. Marys River Entrance Channel," Miscellaneous Paper HL-88-9, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Smith, B. J., Pope, J., and Gorman, L. T. (1994). "Dredging activities and shoaling analysis," *Kings Bay Coastal and Estuarine Physical Monitoring and Evaluation Program: Coastal studies, Volume I: Main text and Appendix A*, Technical Report CERC-94-9, N. C. Kraus, L. T. Gorman, and J. Pope, ed., U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Smith, J. B., Pope, J., Gorman, L. T., Martin, T. R., and Brinson, S. (1994). "Appendix C, Dredging and Shoaling Data," *Kings Bay Coastal and Estuarine Physical Monitoring and Evaluation Program: Coastal studies, Volume II: Appendices B-G*, Technical Report CERC-94-9, N. C. Kraus, L. T. Gorman, and J. Pope (ed.), U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Smith, J. M., Resio, D. T., and Zundel, A. K. (1999). "STWAVE: Steady-state spectral wave model; Report 1: User's manual for STWAVE version 2.0," Instructional Report CHL-99-1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Shore protection manual. (1984). 4th ed., 2 Vol, U.S. Army Engineer Waterways Experiment Station, U.S. Government Printing Office, Washington, DC.

U.S. Army Corps of Engineers. (1961). "Amelia Island, Florida, beach erosion study," House Document No. 200, 87th Congress, 1st Session, U.S. Goverment Printing Office, Washington, DC.

U.S. Army Engineer District, Jacksonville. (1993). "Nassau County, Florida, Fernandina Harbor, section 933 study," Jacksonville, FL.

U.S. Army Engineer Waterways Experiment Station. (1989). "Estimating potential longshore transport rates using WIS data," Coastal Engineering Technical Note CETN-II-19, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS. http://chl.wes.army.mil/library/publications/chetn/#part2

Vemulakonda, S. R., Scheffner, N. W., Earickson, J. A., and Chou, L. W. (1988). "Kings Bay coastal processes numerical model," Technical Report CERC-88-3, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Appendix A
Dredging Data

Appendix A documents all the shoaling and dredging calculations discussed in the main text of this report. Each calculation is presented in the form of a table. The calculations are in the order that they appear in the text. In the tables in this appendix, "MD" denotes maintenance dredging, and "NW" denoted new work.

Table A1
Dredging Records, Compiled from Information Provided by the Jacksonville District

				Disposal Area and Sediment Type (cu yd)							
						Ocean					
Contract #	Year	Type of Work	Total Quantity (cu yd)	Beaches	Nearshore	Sand	Sand/Silt	Silt	Silt/Clay	Unspecified	Cost
	1955	NW	1,305,127							1,305,127	$246,326
	1956	NW	286,545							286,545	$77,141
	1956	NW	1,025,046							1,025,046	$232,017
	1963	MD	319,838							319,838	$67,549
	1964	MD	203,842							203,842	$99,091
	1965	MD	160,967							160,967	$83,761
	1966	MD	178,557							178,557	$136,859
	1967	MD	52,973							52,973	$36,705
	1968	MD	201,000							201,000	$93,555
	1969	MD	180,000							180,000	$59,212
	1969	MD	160,000							160,000	$85,722
	1973	MD	65,000							65,000	$67,689
	1973	MD	411,800							411,800	$142,156
	1974	MD	35,694							35,694	$90,974
	1975	MD	75,915							75,915	$165,405
	1976	MD	108,557							108,557	$302,791
	1979	MD	487,055							487,055	$1,788,401
	1979	NW	450,518							450,518	$9,000,000
	1979	MD	105,000							105,000	$389,514
	1979	MD	620,204							620,204	$1,826,920
	1979	NW	50,647	50,647							$75,000
	1979	MD	224,573							224,573	$826,922
No number	1980	MD	113,784					113,784			$345,000
40-33-989	1982	MD	797,992	359,895		438,097					$2,674,141
No number	1983	MD	78,847	78,847							$308,655
90-34,256	1983	MD	621,884			621,884					$1,925,000
86-C-0028	1986	MD	321,080				321,080				$567,682

(Continued)

Table A1 (Concluded)

Contract #	Year	Type of Work	Total Quantity (cu yd)	Disposal Area and Sediment Type (cu yd)							Cost
						Ocean					
				Beaches	Nearshore	Sand	Sand/Silt	Silt	Silt/Clay	Unspecified	
87-C-0034	1987	NW	906,840	906,840							$3,088,468
87-C-0054	1987	NW	2,148,200		2,148,200						$9,473,022
87-C-0037	1988	NW	5,738,687					5,738,687			$15,853,309
89-C-0002	1989	MD	720,029					720,029			$1,749,059
90-35,479	1989	MD	64,189					64,189			$120,000
90-C-0042	1989	MD	754,104					754,104			$1,560,503
90-C-0100	1990	MD	796,940	147,693	11,121			638,126			$1,252,048
90,34,778	1992	MD	640,237						640,237		$2,238,500
92-C-0010	1992	MD	229,336	193,336					36,000		$2,400,141
90-36,293	1993	MD	253,585					253,585			$1,426,261
93-C-0096	1993	MD	927,230	927,230							$3,905,230
90-36,542	1994	MD	419,060					419,060			$1,887,024
94-C-0026	1994	MD	350,550				350,550				$631,500
90-36,860	1995	MD	183,360					183,360			$15,394
No number	1994	MD	5,000		5,000						$52,000
95-C-0013	1995	MD	254,220	254,220							$2,664,233
96-C-0010	1996	MD	999,956	84,446				915,510			$3,380,980
97-C-0038	1997	MD	437,160		16,579			420,581			$1,190,411
98-C-0003	1998	MD	1,221,404	416,028				805,376			$6,784,361
99-C-0045	1999	MD	402,211	402,211							$5,166,847
99-C-0032	1999	MD	810,636		43,974		766,662				$2,427,320
00-C-0001	2000	MD	1,080,003		568,329			511,675			$3,219,747
01-C-0001	2001	MD	909,341	126,613	391,364	70,872	5,421	385,944			$3,584,815

Table A2 Annual Maintenance Dredging			
Contract #	Year	Type of Work	Total Quantity (cu yd)
	1979	MD	224,573
No number	1980	MD	113,784
40-33-989	1982	MD	797,992
No number	1983	MD	78,847
90-34,256	1983	MD	621,884
86-C-0028	1986	MD	321,080
89-C-0002	1989	MD	720,029
90-35,479	1989	MD	64,189
90-C-0042	1989	MD	754,104
90-C-0100	1990	MD	796,940
90,34,778	1992	MD	640,237
92-C-0010	1992	MD	229,336
90-36,293	1993	MD	253,585
93-C-0096	1993	MD	927,230
90-36,542	1994	MD	419,060
96-C-0026	1994	MD	350,550
90-36,860	1995	MD	183,360
No number	1995	MD	5,000
95-C-0013	1995	MD	254,220
96-C-0010	1996	MD	999,956
97-C-0038	1997	MD	437,160
98-C-0003	1997	MD	1,221,404
99-C-0045	1999	MD	402,211
99-C-0032	1999	MD	810,636
00-C-0001	2000	MD	1,080,003
01-C-0001	2001	MD	909,341
Average Annual Volume=Total/(2001-1979+1)=			**592,031**

Table A3
Percent by Disposal Area

Contract #	Year	Type of Work	Total Quantity (cu yd)	Disposal (cu yd)		
				Beaches	Nearshore	Ocean
	1979	MD	224,573			224,573
No number	1980	MD	113,784			113,784
40-33-989	1982	MD	797,992	359,895		438,097
No number	1983	MD	78,847	78,847		
90-34,256	1983	MD	621,884			621,884
86-C-0028	1986	MD	321,080			321,080
89-C-0002	1989	MD	720,029			720,029
90-35,479	1989	MD	64,189			64,189
90-C-0042	1989	MD	754,104			754,104
90-C-0100	1990	MD	796,940	147,693	11,121	638,126
90,34,778	1992	MD	640,237			640,237
92-C-0010	1992	MD	229,336	193,336		36,000
90-36,293	1993	MD	253,585			253,585
93-C-0096	1993	MD	927,230	927,230		
90-36,542	1994	MD	419,060			419,060
96-C-0026	1994	MD	350,550			350,550
90-36,860	1995	MD	183,360			183,360
No number	1995	MD	5,000		5,000	
95-C-0013	1995	MD	254,220	254,220		
96-C-0010	1996	MD	999,956	84,446		915,510
97-C-0038	1997	MD	437,160		16,579	420,581
98-C-0003	1997	MD	1,221,404	416,028		805,376
99-C-0045	1999	MD	402,211	402,211		
99-C-0032	1999	MD	810,636		43,974	766,662
00-C-0001	2000	MD	1,080,003		568,329	511,675
01-C-0001	2001	MD	909,341	126,613	391,364	391,364
Total=2990519					1,036,367	9,589,826
Percent =22					8	70

Table A4
Number of Years Dredged Since 1955

Contract #	Year	Type of Work	Year Count
	1955	NW	1
	1956	NW	1
	1956	NW	
	1963	MD	1
	1964	MD	1
	1965	MD	1
	1966	MD	1
	1967	MD	1
	1968	MD	1
	1969	MD	1
	1969	MD	
	1973	MD	1
	1973	MD	
	1974	MD	1
	1975	MD	1
	1976	MD	1
	1979	MD	1
	1979	NW	
	1979	MD	
	1979	MD	
	1979	NW	
	1979	MD	
No number	1980	MD	1
40-33,989	1982	MD	1
No number	1983	MD	1
90-34,256	1983	MD	
86-C-0028	1986	MD	1
87-C-0034	1987	NW	1
87-C-0054	1987	NW	
87-C-0037	1988	NW	1
89-C-0002	1989	MD	1
No number	1989	MD	
90-C-0042	1989	MD	
90-C-0100	1990	MD	1
90,34,778	1992	MD	1
92-C-0010	1992	MD	
90-36,293	1993	MD	1
93-C-0096	1993	MD	

(Continued)

Table A4 (Concluded)
Number of Years Dredged Since 1955

Contract #	Year	Type of Work	Year Count
90-36,542	1994	MD	
96-C-0026	1994	MD	1
No number	1994	MD	
90-36,860	1995	MD	1
95-C-0013	1995	MD	
96-C-0010	1996	MD	1
97-C-0038	1997	MD	1
98-C-0003	1997	MD	
99-C-0045	1999	MD	1
99-C-0032	1999	MD	
00-C-0001	2000	MD	1
01-C-0001	2001	MD	1
		# of Years Dredged=	31

Table A5
Number of Years Not Dredged Since 1987

Contract #	Year	Type of Work	Year Count
89-C-0002	1989	MD	
No number	1989	MD	
90-C-0042	1989	MD	1
90-C-0100	1990	MD	1
90,34,778	1992	MD	
92-C-0010	1992	MD	1
90-36,293	1993	MD	
93-C-0096	1993	MD	1
90-36,542	1994	MD	
96-C-0026	1994	MD	
No number	1994	MD	1
90-36,860	1995	MD	
95-C-0013	1995	MD	1
96-C-0010	1996	MD	1
97-C-0038	1997	MD	
98-C-0003	1997	MD	1
99-C-0045	1999	MD	
99-C-0032	1999	MD	1
00-C-0001	2000	MD	1
01-C-0001	2001	MD	1
		# of Years Dredged=	11
		# of Years=(2001-1988+1)=	14
		# of Years not Dredged=	3

Table A6
Annual Maintenance of the Entrance During the 1990's

Contract #	Year	Type of Work	Total Quantity (cu yd)	Cost
90-C-0100	1990	MD	796,940	$1,252,048
90,34,778	1992	MD	640,237	$2,238,500
92-C-0010	1992	MD	229,336	$2,400,141
90-36,293	1993	MD	253,585	$1,426,261
93-C-0096	1993	MD	927,230	$3,905,230
90-36,542	1994	MD	419,060	$1,887,024
94-C-0026	1994	MD	350,550	$631,500
90-36,860	1995	MD	183,360	$15,394
No number	1995	MD	5,000	$52,000
95-C-0013	1995	MD	254,220	$2,664,233
96-C-0010	1996	MD	999,956	$3,380,980
97-C-0038	1997	MD	437,160	$1,190,411
98-C-0003	1997	MD	1,221,404	$6,784,361
99-C-0045	1999	MD	402,211	$5,166,847
99-C-0032	1999	MD	810,636	$2,427,320
		Average MD $ =Sum/(1999-1990+1)=		**$3,542,225**

Table A7
Volume of Dredge Material Classified by Type of Work

Contract #	YEAR	Type of Work	Total Quantity (cu yd)	New Work (cu yd)	Maintenance (cu yd)
	1955	NW	1,305,127	1,305,127	0
	1956	NW	286,545	286,545	0
	1956	NW	1,025,046	1,025,046	0
	1963	MD	319,838	0	319,838
	1964	MD	203,842	0	203,842
	1965	MD	160,967	0	160,967
	1966	MD	178,557	0	178,557
	1967	MD	52,973	0	52,973
	1968	MD	201,000	0	201,000
	1969	MD	180,000	0	180,000
	1969	MD	160,000	0	160,000
	1973	MD	65,000	0	65,000
	1973	MD	411,800	0	411,800
	1974	MD	35,694	0	35,694
	1975	MD	75,915	0	75,915
	1976	MD	108,557	0	108,557
	1979	MD	487,055	0	487,055
	1979	NW	450,518	450,518	0
	1979	MD	105,000	0	105,000
	1979	MD	620,204	0	620,204
	1979	NW	50,647	50,647	0
	1979	MD	224,573	0	224,573
No number	1980	MD	113,784	0	113,784
No number	1982	MD	797,992	0	797,992
No number	1983	MD	78,847	0	78,847
No number	1984	MD	621,884	0	621,884
86-C-0028	1987	MD	321,080	0	321,080
87-C-0034	1987	NW	906,840	906,840	0
87-C-0054	1987	NW	2,148,200	2,148,200	0
87-C-0037	1988	NW	5,738,687	5,738,687	0
89-C-0002	1989	MD	720,029	0	720,029
No number	1989	MD	64,189	0	64,189
90-C-0042	1990	MD	754,104	0	754,104
90-C-0100	1991	MD	796,940	0	796,940
92-C-0010	1992	MD	229,336	0	229,336

(Continued)

Table A7 (Concluded)					
Contract #	Year	Type of Work	Total Quantity (cu yd)	New Work (cu yd)	Maintenance (cu yd)
No number	1993	MD	253,585	0	253,585
93-C-0096	1994	MD	927,230	0	927,230
No number	1994	MD	419,060	0	419,060
96-C-0026	1994	MD	350,550	0	350,550
No number	1995	MD	183,360	0	183,360
No number	1995	MD	5,000	0	5,000
95-C-0013	1995	MD	254,220	0	254,220
96-C-0010	1996	MD	999,956	0	999,956
97-C-0038	1997	MD	437,160	0	437,160
98-C-0003	1997	MD	1,221,404	0	1,221,404
99-C-0045	1999	MD	402,211	0	402,211
99-C-0032	1999	MD	810,636	0	810,636
00-C-0001	2000	MD	1,080,003	0	1,080,003
01-C-0001	2001	MD	909,341	0	909,341
		TOTAL=	**28,894,723**	**11,911,610**	**16,983,113**

Table A8
Average Maintenance Volume before 1987

Contract #	Year	Type of Work	Maintenance (cu yd)
	1963	MD	319,838
	1964	MD	203,842
	1965	MD	160,967
	1966	MD	178,557
	1967	MD	52,973
	1968	MD	201,000
	1969	MD	180,000
	1969	MD	160,000
	1973	MD	65,000
	1973	MD	411,800
	1974	MD	35,694
	1975	MD	75,915
	1976	MD	108,557
	1979	MD	487,055
	1979	MD	105,000
	1979	MD	620,204
	1979	MD	224,573
No number	1980	MD	113,784
40-33,989	1982	MD	797,992
No number	1983	MD	78,847
90-34,256	1983	MD	621,884
86-C-0028	1986	MD	321,080
		Average before 1987=	**230,190**

Table A9
Average Maintenance Volume after 1987

Contract #	YEAR	Type of Work	Total Quantity (cu yd)	Maintenance (cu yd)
89-C-0002	1989	MD	720,029	720,029
No number	1989	MD	64,189	64,189
90-C-0042	1989	MD	754,104	754,104
90-C-0100	1990	MD	796,940	796,940
90,34,778	1992	MD	640,237	640,237
92-C-0010	1992	MD	229,336	229,336
90-36,293	1993	MD	253,585	253,585
93-C-0096	1993	MD	927,230	927,230
90-36,542	1994	MD	419,060	419,060
96-C-0026	1994	MD	350,550	350,550
No number	1994	MD	183,360	183,360
90-36,860	1995	MD	5,000	5,000
95-C-0013	1995	MD	254,220	254,220
96-C-0010	1996	MD	999,956	999,956
97-C-0038	1997	MD	437,160	437,160
98-C-0003	1997	MD	1,221,404	1,221,404
99-C-0045	1999	MD	402,211	402,211
99-C-0032	1999	MD	810,636	810,636
00-C-0001	2000	MD	1,080,003	1,080,003
01-C-0001	2001	MD	909,341	909,341
			Average after **1987**=	818,468

Table A10
Volume of 1987 Channel Expansion

$$\left[11\ \text{ft}_{\text{depth}}\right]*\left[98\ \text{ft}_{\text{width}}\right]*\left[9\ \text{miles}_{\text{length}}*\frac{5{,}280\ \text{ft}}{1\ \text{mile}}\right]*\frac{1\ \text{cu yd}}{27\ \text{cu ft}}=1{,}897{,}280\ \text{cu yd}$$

Table A11
Maintenance Dredging Since 1986

Contact #	Year	Type of Work	Maintenance Dredge (cu yd)
86-C-0028	1986	MD	321,080
89-C-0002	1989	MD	720,029
90-35,479	1989	MD	64,189
90-C-0042	1989	MD	754,104
90-C-0100	1990	MD	796,940
90,34,778	1992	MD	640,237
92-C-0010	1992	MD	229,336
90-36,293	1993	MD	253,585
93-C-0096	1993	MD	927,230
90-36,542	1994	MD	419,060
94-C-0026	1994	MD	350,550
90-36,860	1995	MD	183,360
No number	1994	MD	5,000
95-C-0013	1995	MD	254,220
96-C-0010	1996	MD	999,956
97-C-0038	1997	MD	437,160
98-C-0003	1998	MD	1,221,404
99-C-0045	1999	MD	402,211
99-C-0032	1999	MD	810,636
00-C-0001	2000	MD	1,080,003
01-C-0001	2001	MD	909,341
		Total=	**11,779,631**

Table A12
Maintenance Dredging Since 1986, sta 100-340, sta 0-99, and sta 341-500

%All data was loaded from a text file called station.txt
% volume listing the volume of sediment dredged at each station
%lgth=length of dataset
%Start_st=starting sta from dredging records
%End_st=ending sta from dredging records
%temp in an index of the stations in an individual dredging record
%number is the number of stations listed in temp.
%volperstat
%vol=volume removed between start_st and end_st

```
volume=zeros([1 500]);
for i=[1:lgth]
    if (start_st(i) >0)
        temp=[start_st(i):end_st(i)];
        number=end_st(i)-start_st(i)+1;
        volperstat=vol(i)/number;
        volume(temp)=volume(temp)+volperstat;
    end
 end
```

Results
» sum(volume(100:340)) = 10721517.93 or **10,722,000**
» sum(volume(1:99))+sum(volume(341:500)) = 1021459.07 or **1,021,000**

Table A13
Dredged Material Placement Since 1979, by Percent

Contract #	Year	Type of Work	Total Quantity (cu yd)	Disposal (cu yd)		
				Beaches	Nearshore	Ocean
	1979	NW	50,647	50,647		
	1979	MD	224,573			224,573
No number	1980	MD	113,784			113,784
40-33-989	1982	MD	797,992	359,895		438,097
No number	1983	MD	78,847	78,847		
90-34,256	1983	MD	621,884			621,884
86-C-0028	1986	MD	321,080			321,080
87-C-0034	1987	NW	906,840	906,840		
87-C-0054	1987	NW	2,148,200		2,148,200	
87-C-0037	1988	NW	5,738,687			5,738,687
89-C-0002	1989	MD	720,029			720,029
90-35,479	1989	MD	64,189			64,189
90-C-0042	1989	MD	754,104			754,104
90-C-0100	1990	MD	796,940	147,693	11,121	638,126
90,34,778	1992	MD	640,237			640,237
92-C-0010	1992	MD	229,336	193,336		36,000
90-36,293	1993	MD	253,585			253,585
93-C-0096	1993	MD	927,230	927,230		
90-36,542	1994	MD	419,060			419,060
94-C-0026	1994	MD	350,550			350,550
90-36,860	1995	MD	183,360			183,360
No number	1994	MD	5,000		5,000	
95-C-0013	1995	MD	254,220	254,220		
96-C-0010	1996	MD	999,956	84,446		915,510
97-C-0038	1997	MD	437,160		16,579	420,581
98-C-0003	1998	MD	1,221,404	416,028		805,376
99-C-0045	1999	MD	402,211	402,211		
99-C-0032	1999	MD	810,636		43,974	766,662
00-C-0001	2000	MD	1,080,003		568,329	511,675
01-C-0001	2001	MD	909,341	126,613	391,364	391,364
			Percent=	**18**	**14**	**68**

Table A14
Dredge Placement Since 1980, by Sediment Type

Contract #	Year	Type of Work	Quantity (cu yd)	Disposal (cu yd)							
				Beaches	NearShore			Ocean			
				Sand	Silt	Sand /Silt	Sand	Sand	Sand /Silt	Silt	Silt/ Clay
No number	1980	MD	113,784							113,784	
40-33-989	1982	MD	797,992	359,895				438,097			
No number	1983	MD	78,847	78,847							
90-34,256	1983	MD	621,884					621,884			
86-C-0028	1986	MD	321,080						321,080		
87-C-0034	1987	NW	906,840	906,840							
87-C-0054	1987	NW	2,148,200				2,148,200				
87-C-0037	1988	NW	5,738,687							5,738,687	
89-C-0002	1989	MD	720,029							720,029	
90-35,479	1989	MD	64,189							64,189	
90-C-0042	1989	MD	754,104							754,104	
90-C-0100	1990	MD	796,940	147,693			11,121			638,126	
90,34,778	1992	MD	640,237								640,237
92-C-0010	1992	MD	229,336	193,336							36,000
90-36,293	1993	MD	253,585							253,585	
93-C-0096	1993	MD	927,230	927,230							
90-36,542	1994	MD	419,060							419,060	
94-C-0026	1994	MD	350,550						350,550		
90-36,860	1995	MD	183,360							183,360	
No number	1994	MD	5,000				5,000				
95-C-0013	1995	MD	254,220	254,220							
96-C-0010	1996	MD	999,956	84,446						915,510	
97-C-0038	1997	MD	437,160				16,579			420,581	
98-C-0003	1998	MD	1,221,404	416,028						805,376	
99-C-0045	1999	MD	402,211	402,211							
99-C-0032	1999	MD	810,636			20,885	23,089		766,662		
00-C-0001	2000	MD	1,080,003		511,675		56,654			511,675	
01-C-0001	2001	MD	909,341	55,741	385,944	5,421		70,872	5,421	385,944	
Percent of Sed Type in Specific Disposal Area=				100	28	1	71	7	10	79	4
Percent Disposal of Sand=				53	31			16			
Percent Disposal of Silt=				0	7			93			
Percent Disposal in Specific location=				17.2	14.4			68.4			
Volume of Disposal in Specific location=				3,826,487	3,184,567			15,174,812			
Total Volume Since 1980=				22,185,865							

Table A15
Maximum/Minimum Elevation Change, Outer Channel

12,000 cu yd/year/200 ft is from Table 5.

$$12{,}000 \frac{\text{cu yd}}{\text{year*200 ft (length)}} * \frac{27\,\text{ft}^3}{\text{cu yd}} * \frac{1}{500\text{ ft}} = 3.24\text{ ft/year}$$

Table A16
Time Delay Due to 1 ft of Overdredge

$$\frac{\text{year}}{3.0\text{ ft}} * 1\text{ ft} * \frac{12\text{ months}}{1\text{ year}} = 4\text{ months}$$

Table A17
Volume Required to Overdredge the Outer Channel by 3 ft

$$\left[(sta\ 340\text{- sta }225)*100\text{ ft}\right]_{\text{length}} * 500\text{ ft}_{\text{width}} * 3\text{ ft}_{\text{depth}} * \frac{\text{cu yd}}{27\text{ cu ft}} = 638{,}888$$

Table A18
Maximum/Minimum Elevation Change, Outer Channel

5,000 cu yd/year/200 ft and 20,000 cu yd/year/200 ft are from Table 5.

$$5000 \frac{\text{cu yd}}{\text{year*200 ft (length)}} * \frac{27\,\text{ft}^3}{\text{cu yd}} * \frac{1}{500\text{ ft}} = 1.35\text{ ft/year}$$

$$20000 \frac{\text{cu yd}}{\text{year*200 ft (length)}} * \frac{27\,\text{ft}^2}{\text{cu yd}} * \frac{1}{500\text{ ft}} = 5.4\text{ ft/year}$$

Round to nearest half foot, 1.5 and 5.5 ft/year.

Table A19
Time Between Dredging Events

$$\frac{\text{year}}{3.0\text{ ft}} * 1.5\text{ ft} * \frac{12\text{ months}}{1\text{ year}} = 6\text{ months}$$

$$\frac{\text{year}}{3.0\text{ ft}} * 5.5\text{ ft} * \frac{12\text{ months}}{1\text{ year}} = 22\text{ months}$$

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14. ABSTRACT
This report presents an analysis of the sediment shoaling characteristics of St. Marys Entrance, Florida, performed for the U.S. Army Engineer District, Jacksonville. Shoaling characteristics and alternatives were investigated for the entire inlet entrance channel. Diagnostic Modeling System procedures were applied to determine causes of channel shoaling and evaluate possible mitigation alternatives to reduce the frequency and cost of dredging. Nearshore wave and tidal circulation modeling was conducted to establish patterns of the current in response to channel configuration alternatives and to estimate sediment transport toward the north jetty from the adjacent beach. Morphological analysis based upon bathymetric surveys, combined with modeling results, indicated that eastward extension of the channel widener would increase the time interval between maintenance dredging events.

15. SUBJECT TERMS
See reverse

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15. SUBJECT TERMS

Circulation model
Diagnostic Modeling System
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Florida
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Navigation channel
St. Marys Entrance
Sediment shoaling
Surface-water Modeling System
Tidal inlet
Wave model